Design of concrete structures for durability

Example: Chloride penetration in the lining of a bored tunnel

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The present design method for durability of concrete is based on a set of rules that do not give objective insight in the service life to be expected from a concrete structure. Therefore an objective comparison between different durability measures is not possible. Especially if the lack of durability can lead to loss of human life and high economic losses, this situation is not acceptable. In addition, lack of serviceability and premature repair are not acceptable if methods are available to avoid them.

Designing for a distinct service life means that we have to define all relevant performances that the structure has to fulfil and that can be influenced by degradations. Further we have to define the probability (reliability) that a given performance must be delivered within the design service life. In structural design this approach has been already developed and is followed in practice. That approach is characterised by keywords like performance, ultimate limit state, serviceability limit state and reliability index. To a large extent this approach can be adopted for a performance based service life design.

One of the consequences of the required reliability in the service life design of a structure is the fact that between the design service life and the mean service life a margin must be present. This margin depends on the required level of reliability, the type of service life distribution and its mean value and scatter. Some examples for this margin have been presented in this paper.

In an example of a reinforced concrete lining of a bored tunnel it has been demonstrated how the service life design should be approached. The example shows clearly that the quantified performance approach gives a stronger base for service life design than the conventional approach of durability.

Key words: **concrete, design, durability, service life, reliability, chloride, corrosion**

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1 Present design approach of durability

The present design approach with respect to durability of concrete structures is based on a reasonable understanding of the main degradation processes for concrete, reinforcement and prestressing steel. The performance of the design is however not explicitly formulated as a service life. It is based on deem-to-satisfy rules (for example minimum cover, maximum water/binder ratio, and crack width limitation) and the assumption that if these rules are met, the structure will achieve an acceptably long but unspecified life. The information about the service life to be achieved is to a large extent empirical. Improving the durability increases building costs without any quantification of the reduction of maintenance costs or failure costs. Current design methods only permit to calculate the whole life cycle costs from assumptions with respect to maintenance and failure rates. There are thus no objective means for demonstrating that future maintenance and repair costs will be acceptably low.

This common design approach to durability has other disadvantages. The rules are inadequate in some aggressive environments, while they are too rigorous in other environments. In some cases, this results in a 'belts and braces' approach (many different types of measures on top of each other) which may contain unnecessary and even counteractive measures.

Lack of durability can cause serious safety and serviceability problems for structures. Despite this, usually designers have considerably more attention for load and resistance based structural design than for durability design. Recent history has however shown that due to a lack of durability, serious collapses and other types of failure may occur with large amounts of damage.

On 4 December 1984 at 7 o'clock in the morning the Ynys-y-Gwas bridge in the neighbourhood of Port Talbot in Wales (UK) collapsed [Woodward 1988]. This concrete bridge was a single span segmental post-tensioned structure. Its span was about 18 meters. The bridge was constructed in 1953 and carried a minor road. When its deck collapsed there was no traffic on the bridge. The cause of the collapse was serious corrosion of the post-tensioned tendons. The corrosion took place at the transverse joints in which chloride-containing water could penetrate. This penetration was possible as a consequence of a number of factors, such as the lack of a slab over the beams, ineffective waterproofing, inadequate protection of the tendons, opening of the gap between the segments under live load, poor workmanship and the damp environment over the river.

In Berlin the southern outer roof of the Congress Hall collapsed on 21 May 1980 due to hydrogen-induced stress corrosion [Schaich 1980]. One person died and another was badly injured. The outer roof was characterised by a reinforced concrete arch, which was hollow. Post tensioned steel tendons, which were installed in roof sheeting served as the anchorage of the arch against an inner ring beam. In a part of the roof the sheeting broke away from the ring beam. In another part the sheeting broke away at the outer arch. Corrosion could occur due to design faults leading to cracks in the joints in combination with a porous mortar in the prestressing ducts.

In 1990 a reinforced concrete gallery in Wormerveer (NL) collapsed. This was caused by chloride induced corrosion. Due to poor construction the main reinforcement in the gallery was situated in the lower part in stead of the upper part of the slab. This resulted in a crack where de-icing salts could penetrate. In Melle (B) a prestressed concrete bridge collapsed during the passage of a tank-car filled with gas. The driver died. The cause was a crack that opened during the passage of high loads. Chloride could penetrate through this crack and reach the post-tensioned tendons.

The presented list of structures in which serious accidents occurred as a consequence of insufficient durability demonstrates that the design for durability can be as important as the common structural design. Lack of durability may also lead to other types of failures, leading to a reduced serviceability of the structure and to premature maintenance, repair or even demolishing.

2 Service life design approach

A design method based on establishing the service life of a concrete structure will provide a better design tool than the present method. A complication that becomes explicit if we design for a certain lifetime is the aspect that the service life of a structure cannot be represented by one single value. It is not a deterministic value but it is a stochastic value. That means that it is only possible to describe the service life in terms of stochastics (statistics): the service life has to be presented by means of (for example) a distribution function or a probability density function. Known types of these distribution functions are the normal (Gaussian), lognormal or the Weibull distribution. The main parameters of such distributions are the mean value μ (magnitude parameter) and the standard deviation σ (parameter for the scatter). In stead of the standard deviation we can also use the variation coefficient $V = \sigma/\mu$. In figure 1 an arbitrary example is given for the service life distribution. By means of the reliability index β the probability that the service life is lower than L_1 is identified.

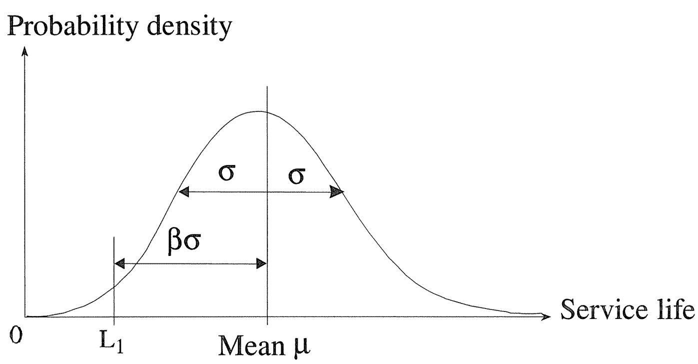

Fig. 1. Example of a probability density function for the service life.

Every structure should fulfil several performances (and functions), to prevent adverse events such as:

- collapse
- overturning, tilting
- extreme vibrations
- high deflections
- out of plumbness
- leakage
- etc.

With every performance a distribution function is connected, depending on the geometry of the structure, the material properties, the mechanical loads and the environmental actions. This implies that for every relevant performance the distribution function has to be established, the design service life as well as the reliability index must be defined. In this respect it is important to define a limit for the performance that separates the adverse state form the required state.

For structural performances the reliability index is in general defined in de building codes.
A distinction is made between:

- the ultimate limit state (ULS), that refers to irreversible adverse events such as loss of equilibrium, collapse, overturning etc,; the ULS is in general connected with high economic losses and/or with loss of human lives
- the serviceability limit state (SLS), that refers to events that restrict the normal use of the structure, such as vibrations or deflections; the economic loss is restricted and victims are not to be expected.

In table 1 some values for the reliability index β are given. These reliability indexes can be used if lack of durability leads to an event that is similar to loss of structural safety [Siemes and Rostam, 1996]. In the table the approximate failure probability corresponding with the reliability index is also given. For example: for the consequences (economic loss or victims) it is not relevant if collapse is due to overloading or due to excessive material degradation. For non-structural performances it will be necessary to define reliability indexes. If failure leads only to economical consequences, the reliability index can be based on an economic optimisation (based on a balance between the extra costs for improving the durability by design or by maintenance and the reduction of the risk = failure probability multiplied with the expected failure costs).

Table 1. Examples of some reliability indexes in structural codes.

	Reliability index β in a period of 50 year		Approximate failure
type of performance	EuroCode	Dutch Building Decree	probability in 50 year
ULS	3.8	3.6	10^{-4}
SLS	1.5	1.8	10^{-2}

In figure 2 and 3 the consequences of the reliability indexes from the Dutch Building Decree are given in a service life distribution for respectively the SLS and the ULS. In this example it has been assumed that the service life distribution is lognormal with a variation coefficient of 30 % ($s = 0.3\ \mu$). These assumptions are more or less realistic for this kind of durability problems.
To illustrate the effect of the value for the design service life, the figures are given for 50 year and 100 year, leading to mean service lifes m_L of 88,5 year and 177 year for the SLS for design lifes of 50 year and 100 year, respectively. For the ULS μ_L is 150 year and 300 year, respectively. From this example we learn that we must design with a relatively high margin γ_L between the mean service life and the design service life. In table 2 this has been quantified. The margin γ_L depends on the type of distribution and the variation coefficient

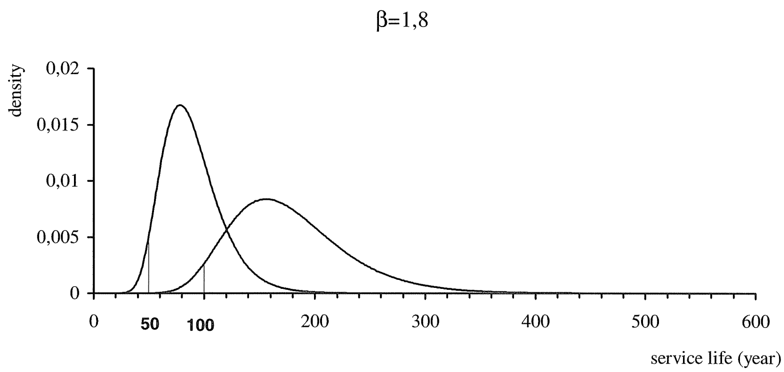

Fig. 2. Example of a service life distribution for the SLS ($\beta = 1.8$) for a design period of 50 year and for a design period of 100 year. (A lognormal distribution with a coefficient of variation of 0.3 has been assumed).

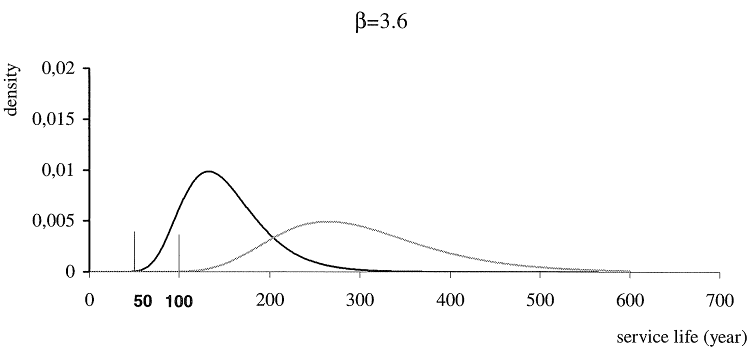

Fig. 3. Example of a service life distribution for the ULS ($\beta = 3.6$) for a design period of 50 year and for a design period of 100 year. (A lognormal distribution with a coefficient of variation of 0.3 has been assumed).

In principle we do not know the probability distributions for the performances of a concrete structure. They will have to be calculated using probabilistic methods. Some examples can be found in [Siemes and Vrouwenvelder 1985; Sarja et al 1996 and Schiebl et al 1997]. Recently the Brite/EuRam project DuraCrete has been completed, giving a first manual for service life design of concrete structures [DuraCrete 1999].

Table 2. Mean service lifes and margins γ_L for design service lifes of 50 year and 100 year.

Type of performance	Design life 50 year		Design life 100 year	
	Mean service life	Margin γ_L	Mean service life	Margin γ_L
SLS: $\beta = 1.8$	88.5	1.8	177	1.8
ULS: $\beta = 3.6$	150	3.0	300	3.0

3 Example: design of a tunnel lining exposed to chlorides

3.1 *Description of the lining and the environment*

In the Netherlands we have recently started to design and construct bored tunnels in soft soil. Both traffic and railway tunnels are being built. This example is restricted to tunnels transporting motor vehicle traffic. The lining of these tunnels consists of prefabricated reinforced concrete elements (Figure 4). The tunnel rings consist of seven of these elements and one keystone.
The lining has a wall thickness of about 0.4 m. Around each element and keystone a rubber joint profile is applied to secure the watertightness of the tunnel lining. The surfaces of the joints of the elements are provided with notches or recesses to centre the rings and the elements during construction of the lining. In practice these notches and recesses may contribute to the shear strength of the tunnel. The elements and the keystone form a ring with an internal diameter of about 10 m. The tunnel lining consists of a long series of rings (Figure 5).

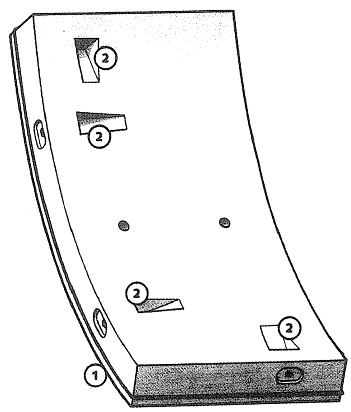

Fig. 4. Element of the tunnel lining.

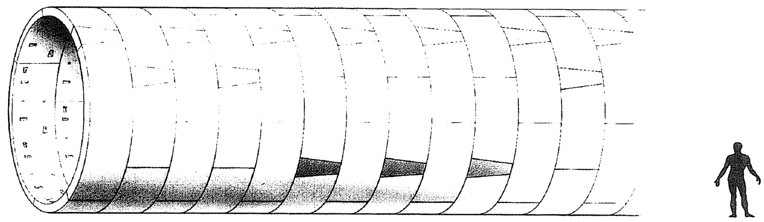

Fig. 5. Tunnel lining consisting of rings composed of elements and keystones.

The tunnel in our example will mainly be subjected to the following environmental actions:

- Carbon dioxide from the atmosphere at all surfaces in contact with the atmosphere. The effect of this carbon dioxide is important to consider during design, as the amount of carbon dioxide in the tunnel will be raised with respect to the outdoor environment with approximately a factor 2 due to emissions from the traffic and the low ventilation in the tunnel. Further there will be no precipitation in the tunnel. Blockage of the pores in the concrete cover due to precipitation will therefor not occur. Condensation may compensate that partly. The water that is emitted by motor vehicles will influence condensation.
- De-icing salts. In the Dutch climate the number of times and the duration that the tunnel tubes will be exposed to frost is limited. The entrances of the tunnel however will be exposed to frost every time that frost will occur outside the tunnel. Further we may expect that vehicles will transport de-icing salts from the road outside into the tunnel tubes. The de-icing salts have two effects on the durability of the tunnel lining:
  1. scaling of the surfaces exposed to frost and de-icing salts
  2. chloride ingress into the concrete leading to initiation of corrosion; in the Netherlands de-icing salts typically consists of calcium chloride or sodium chloride.
- Chloride ingress from the salt ground water. In parts of the Netherlands the chloride content of the ground water is similar to the chloride content of seawater. The saline ground water will penetrate in the outer part of the elements (concrete outside the rubber joints). Due to possible leakage of the joints (immediately after construction or during the service life), the saline ground water will also penetrate in the joints at the inner side of the lining. The amount of saline ground water and the leakage spots can hardly be predicted.
- Stray currents. They may come from existing railroads, cathodic protection systems for steel pipelines in the neighbourhood of the tunnel and so on. Due to all kinds of construction activities the importance of stray currents may grow during the service life of the tunnel.
- Alkali-aggregate reaction may occur if the combination of aggregates and cement is sensitive to this reaction.
- DEF (delayed ettringite formation) may occur if the prefabricated concrete elements are steam cured at a too high temperature.

According to the Dutch concrete code such a tunnel should be designed with a minimum concrete cover of 35 mm and a maximum water/cement ratio of 0.45. Further it is common practice to use blast furnace slag cement in a saline environment. This type of cement also prevents alkali-silica reaction (ASR). The code has not been based on a distinct service life. The general idea is that the service life of most concrete structures is about 50 year (with a low probability of repair in that period). Concrete structures in marine environments have to meet in principle higher requirements, although these have not been defined in the code. Additional protection should be provided, such as using blast furnace slag cement (with a better resistance to chloride penetration). The present codes do not prescribe such measures explicitly. The effect of such additional protective measures cannot be evaluated in an objective and quantified way on basis of the existing codes.

It can be shown that for a tunnel described here as an example, chloride ingress constitutes the most severe threat to the durability of the structure. Therefore, this article focuses on chloride ingress.

3.2 *Chloride ingress in the tunnel segments*

For this example we will concentrate on the effects of the chloride ingress into the concrete. Only the initiation time (until the onset of corrosion) will be considered. For a complete durability design it will also be necessary to take into account the corrosion phase both for the SLS and the ULS.

In figure 6 a cross section of the tunnel is given, indicating three different zones for the ingress of chlorides:

1. splash zone; this zone will be loaded by de-icing salt spray or by saline leakage water in the joints
2. tidal zone; water that enters the tunnel tubes will be collected in the lower part of the cross section; if the level of this water is high enough it will be pumped out via drainage tubes in the lower part
3. submerged zone; under the level of the drainage tubes there will always be water present at the inner side of the tunnel.

The names of these zones refer to a seawater environment. Most of the knowledge that we have on the ingress of chlorides originates from sea climates.

As can be seen from figure 6 an essential part of the inside of the tube will be inaccessible for inspection and repair due to the presence of the road. For tunnels with a fire protection lining the remaining (upper) part will also be inaccessible for direct inspection. It is obvious that the outside of the tunnel lining is also inaccessible. This means that the tunnel lining should in principle be designed as a 'maintenance-free' structure.

The chloride concentration due to the use of de-icing salts is uncertain but it may become relatively high. The uncertainty is caused by the fact that the amount of de-icing salts to be used in the tunnel is hardly known. The amount of water coming from precipitation will be limited, as at the entrance of the tunnel tube a drainage system will prevent it from entering the tunnel. The chloride content may even become higher than in seawater.

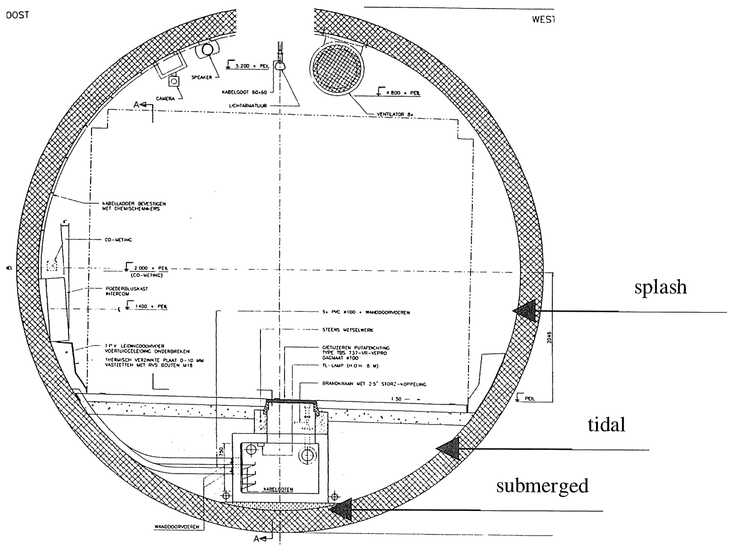

Fig. 6. Cross section of the tunnel tube with the various chloride-exposed zones.

The calculations will be based on the following considerations:
- the chloride content in the submerged zone and in the tidal zone will be 20 g/l, comparable to sea-water (this estimate is too optimistic if the fresh water diluting the chlorides only originates from the limited precipitation inside the tunnel)
- diffusion will be taken as the transport mechanism; although the authors realise that pure diffusion only occurs in saturated concrete, this must be accepted because chloride transport in non-saturated conditions cannot be modelled with reasonable accuracy
- the chloride content in the splash zone is comparable to those reported from a test site near a highway in the UK [Bamforth 1997]
- if the chloride content near the reinforcement reaches a critical value C_{cr}, corrosion will be initiated; this will be considered as a limit state that shall not be exceeded within the design service life in order to assure that the structure will be maintenance free; C_{cr} will be taken as 1.0 % by mass of cement, which is realistic for high concrete qualities [Polder 1996] ; for carbonated concrete this value will be too optimistic
- the reliability index for this case will be set to 1.8 as it will be related to a SLS
- the depth of the concrete cover is 50 mm except for the joints between the elements, because of the high splitting stresses during the boring process this value was restricted to 20 mm.

Diffusion of chlorides can be modelled using Fick's second law of diffusion:

$$C_x(t) = C_S\left(1 - erf\left(\frac{x}{2\sqrt{D_{Cl}t}}\right)\right)$$

with:

C_x = chloride content at a depth x after an exposure time t

C_s = surface chloride content; this will be taken as 3 % or 4 % by mass of cement, being a normal value for marine structures and also near highways (Bamforth, 1997)

erf = error function

D_{Cl} = diffusion coefficient for chloride penetration.

For this paper, a slightly simplified equation for the time until initiation L_i according to [Polder 1996] will be used:

$$L_i = d^2/(A \bullet D_{Cl})$$

with:

L_i = time until initiation of corrosion

d = depth of concrete cover

C_s = surface chloride content

C_{cr} = critical chloride content

A = constant depending of C_s and C_{cr}; using Fick's law it can be calculated that $A = 1.8$ for $C_{cr} = 1$ % and $C_s = 3$ %; $A = 2.65$ for $C_s = 4$ % and $C_{cr} = 1$ % [Polder 1996].

If in the above formula the mean values of all the variables are substituted, the result (initiation time) will approximately be a mean value. This value should be compared with the mean values in table 2 for the SLS if the start of the propagation phase is considered as the SLS. For a design service life of 100 year the mean value should be at least 177 year (in case of a lognormal distribution with a coefficient of variation of 0.3).
At first calculations have been made using a diffusion coefficient $D_{Cl} = 1{,}5\ 10^{-12}$ m^2/s, being a realistic value for a dense Portland cement concrete with a low water/cement ratio. The results of the calculation are presented in Table 3. A low diffusion coefficient does not only depend on the mix composition but also on the quality of pouring, compacting and curing of the concrete.

Table 3. Mean initiation time (= mean service life for maintenance free concrete) for $D_{Cl} = 1.5\ 10^{-12}$ for various concrete covers; $A = 2.65$ for $C_s = 4$ %; $A = 1.8$ for $C_s = 3$ %.

D_{Cl} [m^2/s]	A	d [mm]	L_i [year]
$1.5\ 10^{-12}$	2.65	50	20
		35	10
		20	3
		15	2
	1.8	50	30
		35	14
		20	5
		15	3

The mean initiation time in this calculation for a concrete cover of 50 mm is 20 year or 30 year, depending on the value for A. For a reliability index $\beta = 1.8$ we need however a mean initiation time of 177 year (according to table 2). It is clear that the design of the tunnel lining has to be improved. In Table 4 the calculation has been made for concretes with lower diffusion coefficients. Values of $7.5\ 10^{-13}$ m^2/s and $3.0\ 10^{-13}$ m^2/s have been added. These values represent blast furnace slag cement concretes with a high slag content or fly ash cement concrete with high quality ash and a high fly ash content [Polder 1996, Polder & Larbi, 1995]. The calculation results show that even with the lowest diffusion coefficient and the favourable value of $A = 1.8$, the mean initiation time is 148 year. All other values for L_i are far too low to be acceptable. An initiation time of 148 year is however close to the value 177 year that is required.

Table 4. Mean initiation time for a concrete cover of 50 mm and for various chloride diffusion coefficients.

D_{Cl} (m^2/s)	A	d (mm)	L_i (year)
$1.5\ 10^{-12}$	2.65	50	20
	1.80	50	30
$7.5\ 10^{-13}$	2.65	50	40
	1.80	50	59
$3.0\ 10^{-13}$	2.65	50	100
	1.80	50	148

In figure 7 an impression has been given of the penetration of chlorides into the concrete after a period of 100 year for the concretes from table 4. We see that after 100 year the concrete with $D_{Cl} = 3.0\ 10^{-13}$ m^2/s at a depth of 50 mm will contain 1 % chloride by mass of cement. Because the figure is based on mean values, there is a probability of 50 % that the chlorides will reach the reinforcement with a cover of 50 mm within the period of 100 year. To achieve the same result the other mentioned concretes (with higher D_{Cl}) even need much higher cover thicknesses.

In the calculations we have assumed a distinct surface chloride content. In literature the actual value of the surface content is still under discussion. To evaluate the effect of the surface content some calculations have been made with various surface contents. The results have been presented in figure 8. It is clear from the figure that if the reinforcement has a thick concrete cover the influence of the surface content is restricted, especially if the concrete is very dense for chlorides. However the chloride content after 100 year at a cover thickness of 50 mm differs in this example still by almost a factor of 2.

For designing the thickness of the concrete cover we can refer to figure 9. This figure is based on concrete with a chloride diffusion coefficient $D_{Cl} = 0.3\ 10^{-12}$ m^2/s. Diffusion lines are given for periods of 50 year, 100 year, 177 year and 300 year. As failure has been defined as the end of the initiation phase or a critical chloride content 1 % at the reinforcement the corresponding minimum covers are 35, 50, 67 and 87 mm. In table 5 an overview of these results has been given.

As stated before according to the Dutch Concrete Code a cover of 35 mm will be sufficient for a period of approximately 50 year. This implies however a failure probability of 50 %. To get this same failure probability in a design period of 100 year the concrete cover needs to be 50 mm.
To get a failure probability of 1 % (as in an SLS) the concrete cover should be 67 mm. For a failure probability of 0.01 % (as in an ULS) the concrete cover should be 87 mm. It is however obvious that the end of the initiation phase is in general never defined as an ULS. The figure 0.01 % has only been given for comparison.

Table 5. Minimum cover for various service lifes; the end of service life (or failure) being defined as the end of the initiation phase.

Mean service life	Concrete cover	Comment
50 year	35 mm	50 % failure in 50 year
100 year	50 mm	50 % failure in 100 year
177 year	67 mm	1 % failure in 100 year
300 year	87 mm	0.01 % failure in 100 year

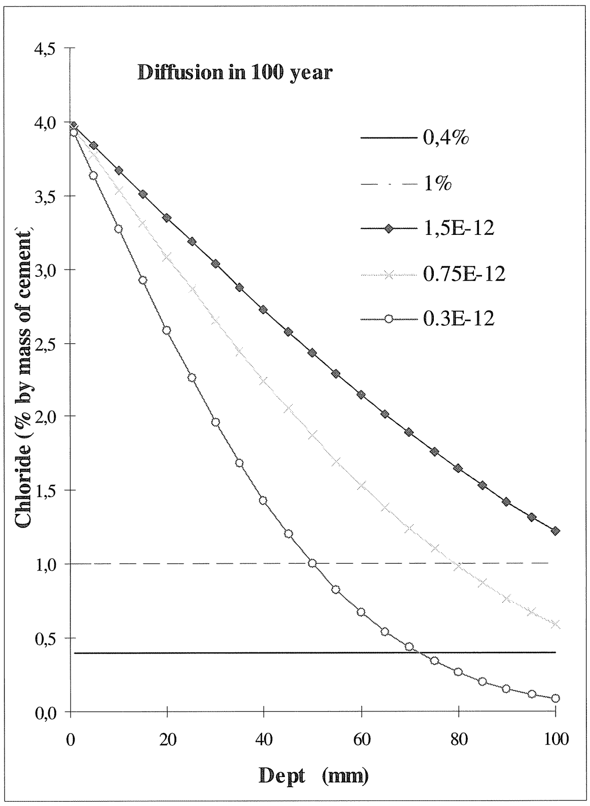

Fig. 7. Chloride penetration after 100 year for concretes with various diffusion coefficients and a surface chloride concentration of 4 %; chloride levels of 0.4% and 1% indicated.

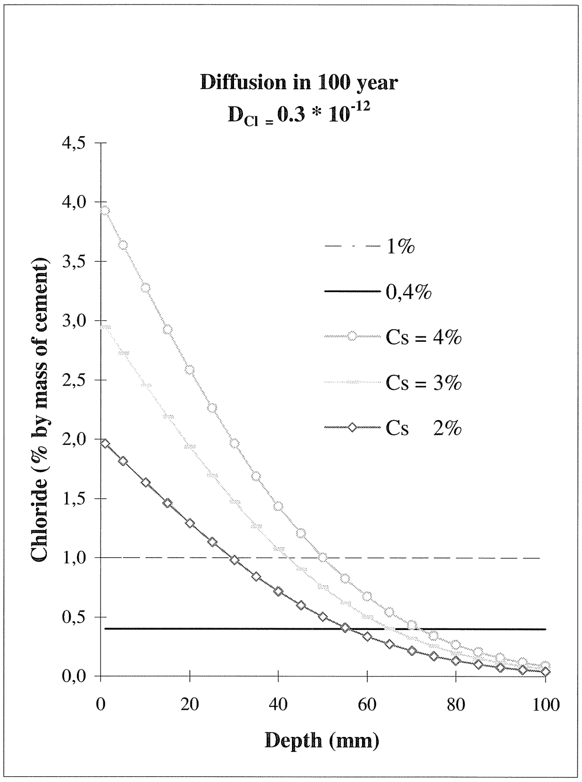

Fig. 8. The effect of the surface chloride concentration on the chloride profile after 100 years for a low value of the diffusion coefficient.

In the calculation examples we have based the conclusions on the mean values for the service life. This is however a simplification mainly meant to have a first impression for checking the design, without making complicated probabilistic calculations. For a realistic design it will in general be necessary to make a full probabilistic calculation. In the near future it may be expected that simplified methods with probabilistic underpinning will be developed that are reliable to use in the design phase.

Moreover in the presented example the effect of corrosion has not been taken into account.
For a complete and realistic design this should be done. Very severe corrosion may even lead to exceeding an ULS, where a higher reliability index ($\beta = 3.6$) is required. Even if the SLS (or maintenance) requirements are fulfilled, the other performances both for the SLS and the ULS have to be checked.

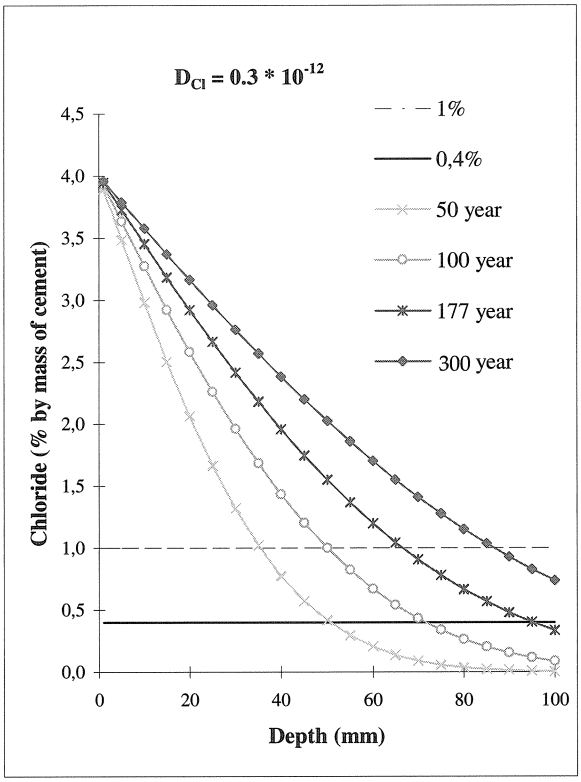

Fig. 9. Chloride [profiles] after 50, 100, 177 and 300 years for a low value of the diffusion coefficient.

3.3 Evaluation of the results

The durability evaluation has shown that the conventional design approach may not provide the required service life of 100 year with sufficient reliability. This is the case in particular for cover depths up to 50 mm (in chloride contaminated environment). The calculation has been based on reasonable or even mild values for the surface and critical chloride contents, and using the lowest chloride diffusion coefficient found in our practice. Consequently, additional or alternative measures have to be considered to improve the durability. Ideally, their effect on the service life should be quantified in terms of the obtained reduction of failure probabilities. Because the required information is not available in many cases (and in particular not in appropriately quantified form), such calculations will be difficult. At present, the best option is surveying literature and / or seeking expert opinion for evaluating various alternative measures. Examples of such measures will be given here and discussed briefly.

Additional measures may include improving the concrete quality or using higher concrete cover. With our present knowledge it seems very difficult to improve the concrete quality in a reliable way to the level where the desired result can be obtained. Increasing the concrete cover beyond 50 mm is practically impossible. For instance, in the elements of a bored tunnel a very high cover (unreinforced zone) will increase the probability of cracking due to jacking forces during tunnel construction. Such cracking would result in leakage of the tunnel and in corrosion of the reinforcement. At present, these two conventional measures do not offer a suitable solution.

Corrosion of reinforcement could be prevented using alternative construction methods. Such preventative measures should result in a lower probability of corrosion initiation. Examples of such measures are [COST 1997]:
- stainless steel reinforcement (at least at the threatened spots)
- reinforcement with organic or metallic coatings
- adding corrosion inhibitors to the concrete mix
- hydrophobic treatment of the threatened concrete surfaces
- applying a coating to the threatened concrete surfaces
- cathodic prevention.

Stainless steel offers a higher tolerance for chloride, of the order of several percent by mass of cement [COST 1997]. A practical and economic design would use stainless steel at the most threatened areas, for about 10 or 20 % of the total reinforcement. The service life has been estimated for concrete with stainless steel. Assumptions are: surface chloride 4 %, critical chloride 2.5 % chloride (stainless steel 316 at welds), 177 year (mean) required for SLS. The minimum required cover depth to obtain this would be 30 mm. Using stainless steel obviously reduces the required cover depth necessary to prevent corrosion initiation in 100 year in a reliable way. The result could be acceptable for the tunnel in the example discussed.

Coating the reinforcement is not considered sufficiently reliable. Organic coatings on reinforcement have been used for some time in the USA. However, the experience is not very positive: corrosion initiation continues to occur relatively quickly and the improvement of service life is negligible [Sagues 1995]. Metallic (zinc) coatings have been reported to increase the tolerance for chloride, but only to a limited extent. Whether the required service life improvement can be obtained is questionable.

Adding a corrosion inhibitor to the fresh concrete mix has been applied in practice in the USA on a reasonable scale and some new products have appeared recently. At least for calcium nitrite, the mitigating effect on corrosion is clear. Because inhibitors are usually soluble or even volatile, it is less clear if this effect will be durable. The active substances may very well disappear from the steel / concrete interface within the service life before they are needed.

Hydrophobic treatment of the concrete is a protective measure used to a reasonable extent for instance in the United Kingdom and The Netherlands. Hydrophobic treatment has been shown to reduce the penetration of chloride in terms of flow by a factor 3 to 5 [Polder et al. 1996]. The effect remains intact for many years [Vries et al. 1998]. Further calculation is necessary to evaluate the contribution to the desired reliability.

Coating the concrete surface may strongly reduce chloride ingress. However, a coated surface is vulnerable to damage due to mechanical impact. As the construction process involves considerable transport and handling of the elements, some damage inevitably will occur. Besides it must be realised that the service life of a coating is restricted. Proper evaluation of this option should take these aspects into account.

Cathodic prevention provides an alternative protection mechanism by changing the electro-chemistry of the steel / concrete interface. As such it is parallel to the chloride penetration resistance offered by the concrete cover. Having two parallel protection mechanisms will strongly improve the reliability of the protection. Cathodic prevention requires monitoring and regular testing, thereby increasing the overall cost. However, its expectedly strong effect on durability makes this a promising way to ensure a sufficiently long service life.

Using another structural material may provide another possibility, like steel fibre reinforced concrete. This seems to have a higher tolerance for chloride in terms of corrosion initiation and propagation. The experience is not very well documented under practical conditions. At present for the design this option is therefor not yet reliable.

Finally, the durability evaluation for the SLS can be modified by taking into account (a part of) the propagation phase. This would require more complete understanding of the factors controlling the corrosion rate and the effects of corrosion on the performance of the structure. Corrosion of reinforcement could lead to cracking and subsequent leaking of a tunnel. It is unclear, which amount of leakage would occur at which amount of corrosion and what failure probabilities are associated. The models available at present should be improved considerably before such calculations could be made. Corrosion will further lead to a reduction of the load bearing capacity of the elements. For that reason this mechanisms has also to be considered in the ULS.

4 Conclusions

The conventional durability design for concrete structures is in principle too optimistic for a bored concrete tunnel in a saline environment. This type of design does not give options to improve the design in an objective way. The effects of improvements cannot be quantified.

In the conventional approach the concrete cover is more or less considered as a protective coating for the reinforcement. Rules are given for the minimum thickness and quality (water / binder ratio). The rules do not differentiate between serviceability limit states and ultimate limit states. Consequently the reliability for both limit states has not been properly differentiated. This conflicts with logic and with the structural design approach.

The example calculations of the tunnel lining have however made clear that without a probabilistic approach it will not be possible to make an objective service life design. At this moment it is not yet possible to make a realistic probabilistic service life design for concrete structures in general. This is due to a lack of proper models for the SLS and especially the ULS, of data, and of target reliabilities. The development of the service life design method is however proceeding [Sarja et al 1996, Siemes and Rostam 1996, Schießl et al 1997, DuraCrete 1999].

For the time being we have to apply simplified models and partly to rely on opinions of experts. As long as the results are on the safe side this is an improvement compared to the conventional durability design. The calculation examples have shown that making a first rough design is even possible without actually making probabilistic calculations.

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