

Penetration of chloride from de-icing salt into concrete from a 30 year old bridge

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This paper presents the analysis of chloride profiles obtained from a reinforced concrete bridge. It was the bicycle part of the Berkumer bridge near Zwolle, The Netherlands. For about 30 years, this bridge was exposed to de-icing salts. The bicycle lane was coated with an epoxy coating. The parapets (side walks) were left uncoated. A total number of 36 chloride profiles were analysed using a diffusion model. The chloride diffusion coefficient is typical for Portland cement concrete in wet conditions, about $3 \cdot 10^{-12} \text{ m}^2/\text{s}$.

In the cycle-lane part of the bridge, a diffusion coefficient of $3 \cdot 10^{-12} \text{ m}^2/\text{s}$ and surface contents of about 2–4 % describe the penetration of chloride rather accurately. This is unexpected in the presence of an epoxy coating. It appears that the protective effect of the coating was practically lost after about 15 years of service. The presence of the damaged coating has however caused the concrete to remain relatively wet, so allowing quick chloride transport.

The non-coated parapets on the other hand, have not shown quick chloride penetration despite significant chloride loads (as demonstrated by significant surface contents). The calculations give quite low apparent diffusion coefficients for the parapet, which are not in agreement with typical values for Portland cement concrete (assuming near water saturation). Apparently the absence of a coating has allowed the concrete of the parapets to dry out substantially, so slowing down chloride transport.

These results suggest that the protective effects of (epoxy) coatings in these types of structures should be reconsidered. It is suggested to improve the protective effect by applying hydrophobic treatment of the concrete before applying this type of coatings and it may be useful to monitor coating integrity.

There appears to be a need for improved coatings for this type of applications, ideally combining high chloride penetration resistance, high water evaporation, good mechanical resistance and a long effective service life. Alternatively, the cover to the steel should be increased and/or a blast furnace slag cement (with higher resistance to chloride penetration) should be chosen.

Key words: concrete, reinforcement corrosion, chloride, diffusion, bridge, coating

1 Introduction

Normally steel reinforcement in concrete is protected from corrosion by passivity. This is a state of negligible corrosion rate, caused by an atomically thin oxide film on the steel surface, which is stabilised by the high pH in concrete (about 13). This passivation may be lost by two causes: either carbon dioxide ingress, which reduces the pH to values about 9 (carbonation), resulting in a uniform loss of passivation, or the presence of chloride ions, which locally break down the passive

film. Chloride may either be cast in as a set accelerator or penetrate from de-icing salts or sea water. Penetration from de-icing salts is the main subject of this paper. However, we first consider what happens after chloride has initiated corrosion.

Once chloride ions arrive at the steel in such high quantities that passivation is lost, corrosion pits develop. These pits are (initially) small spots where iron passes into solution as ferrous ions (Fe^{2+}), leaving excess electrons in the steel; this reaction is termed anodic. At the steel surface outside the pits, water and oxygen molecules react with the excess electrons to form hydroxide (OH^-) ions; this is called the cathodic reaction. The ferrous ions from the anodes react with hydroxide ions from the cathodes and with more oxygen to form various solid hydrated iron oxides. These corrosion products are commonly called "rust". Part of the iron oxides may remain in solution, migrate to the concrete surface and become visible as brown spots (often taken as a first warning that something undesirable is happening). Corrosion products are more voluminous than the original steel. The net effect is expansion, causing tensile stresses in the concrete cover. After relatively small amounts of steel have been transformed into corrosion products, the concrete cracks and subsequently spalling or delamination occurs. Cracking and spalling are further signs of deterioration. When left to corrode, the steel bar diameter may decrease below structurally acceptable values. Moreover, spalling may disturb the fitness-for-use of a structure, for example a bridge deck. Repair and reinstating corrosion protection may involve high costs and disturbance of traffic. One method used is cathodic protection, see (Polder 1998).

De-icing salts are mainly sodium or calcium chloride (or mixtures). The amount and frequency of application depends on the particular climate. In The Netherlands, winters are usually mild with limited periods of freezing temperatures and some snow. The average amount of salt strewn on bridge surfaces with normal (dense) asphalt may be estimated at about 250 gram chloride per square meter per year. This constitutes an aggressive environment for reinforced concrete; however, in various other countries the amount of salt used is considerably higher.

The principle barrier against de-icing salt penetration is the concrete cover to the steel. It should be as thick as possible and have a high resistance to chloride penetration (see Siemes et al. 1998). Blended cements (containing blast furnace slag and fly ash) have been shown to have a high chloride penetration resistance in marine environment (Polder & Larbi 1995, Polder 1996). Blended cement behaviour under wet/dry cycles simulating de-icing salt load is being studied. In many cases, preventative measures are taken which are additional to the normal concrete cover. One of these additional protective measures is applying a coating. On bridges carrying only light traffic an epoxy coating is applied to the deck. This may be regarded a reasonable compromise between protection, road surface properties and weight. This paper analyses chloride penetration into concrete of a bicycle bridge coated with an epoxy coating of a few millimeters thickness and exposed to de-icing salts during about 30 years service.

2 Description of the structure and the specimens

The concrete described in this paper was obtained from the bicycle part of the Berkumer bridge deck in Zwolle, The Netherlands. The bridge was demolished after about 30 years of service due to severe delamination caused by reinforcement corrosion. A new deck replaced the old one in 1996. The old deck was a reinforced concrete slab (supported by steel beams) of 130 m long and 4.73 m wide, built in 1965. The 160 mm thick concrete deck contained two layers of reinforcement, with a minimum cover of 25 mm (bottom side) or 30 mm (top side). Both of these layers contained rebars in two directions, perpendicular to each other. The actual bicycle carrying part was epoxy coated in order to reduce penetration of water and de-icing salts to the top layer of reinforcement; de-icing salts were used here. The sidewalk (parapet) parts were unprotected; no de-icing salts were applied here directly. Figure 1 shows a cross section of the bridge, showing both the cycleway and the adjacent two-lane motorway. Figure 2 shows the cross-section of the cycle bridge.

Although no detailed data on the use of de-icing salts in the Netherlands are available, a semi-quantitative description is possible. In an average year, de-icing salt is applied about 40 times. On motorway surfaces about 6 gram chloride (ion) per m^2 is applied on normal (dense) asphalt; up to 11 g/m^2 on porous asphalt (ZOAB), each per application; this amounts to an average of 250 to 450 $\text{g}/\text{m}^2/\text{year}$.

During an inspection of the bridge in 1994, cores were taken and subsequently analysed for chloride. The objective was to document the state of the bridge deck and to advise on future maintenance. The inspection involved taking chloride samples and impulse radar surveying. With impulse radar it was found that considerable delamination was present, on about half the total surface of the deck. This was attributed to heavy corrosion of the reinforcement. The owner decided that repair would be too costly as compared to building a new deck. Consequently, the old deck was demolished and a new deck was put in place.

After demolition of the bridge, eight parts were obtained for various tests. These parts were slabs with dimensions between $4.7 * 1.2 \text{ m}^2$ and $4.7 * 3 \text{ m}^2$. From these slabs two specimens have been taken from the sidewalk (parapet) part (the left-hand side of the cross section in Figure 2) and two from the bicycle-carrying surface or cycle-lane part (opposite end of slab in Figure 2). These specimens were used for cathodic protection tests, and were investigated using chloride analysis, resistivity measurements and thin section microscopy in 1996. Preliminary results of cathodic protection tests have been described in [Polder & Nerland 1998]. This paper does not deal with those tests. In later stages dust samples were taken and analysed for chlorides, both in 1996 and 1998. Groups of samples were coded using capitals A through H.

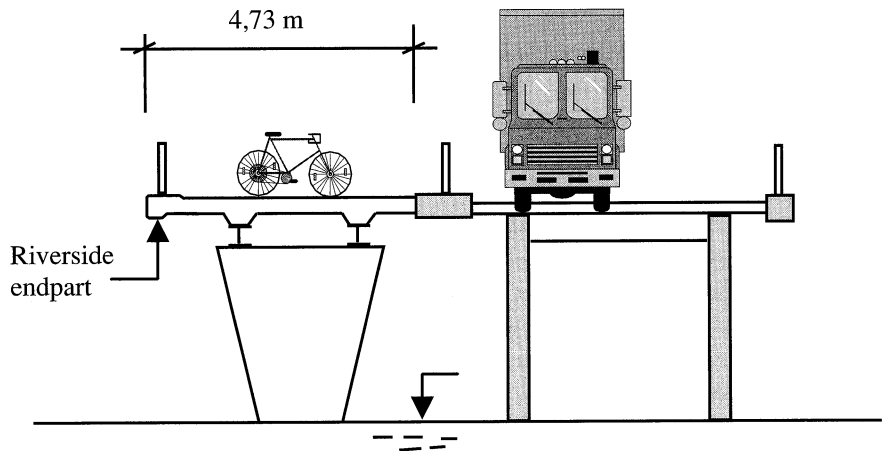


Fig. 1. Simplified cross section of the Berkumer bridge over the river Vecht at Zwolle, showing both the cycleway (investigated here) and the adjacent two-lane motorway.

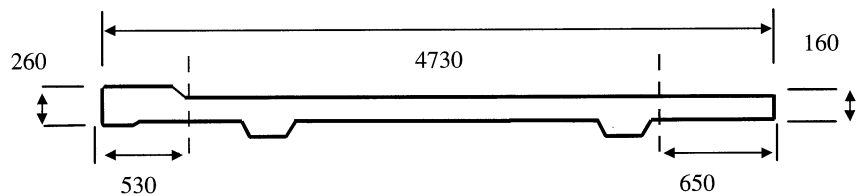


Fig. 2. Cross-section of the bridge deck (sizes in mm), left (thicker part) parapet, not coated, thin part cycle deck, coated.

3 Laboratory testing and results

3.1 Testing

Chloride analysis

Cores with a diameter of 50 mm were taken through the slabs for chloride analysis. They were cut in slices using a water cooled diamond saw. The slices were dried at 105°C overnight and then crushed and ground to powder of maximum grain size about 0.25 mm. The powder was dissolved in hot nitric acid of about 3 M strength. After filtering over paper filters, chloride was analysed for in the liquid by titration according to Volhard. The chloride contents were calculated by weight of cement assuming that 20% of the acid-soluble mass comprised of hydration water. This method of chloride analysis complies to a procedure which was approved by Rijkswaterstraat after an inter-laboratory test scheme [Gulikers et al. 1996]. It was found that the accuracy of individual results following this procedure is about 10% or better [Gulikers 1996]. In addition, dust samples were taken

with a massive drill from holes drilled for reference electrodes and for other purposes. They were analysed for chloride described as above.

Other testing

The carbonation depth of the concrete was measured at different points on the specimens and on former adjacent parts on the slabs with phenolphthalein. The total number of measurement locations was 16 (two on upper and two on lower surface of four slabs each).

For resistivity measurements, the remaining parts of the cores that were drilled for chloride analysis have been used. To study the influence of temperature and relative humidity on resistivity, the cores were exposed to different environmental conditions. They were exposed to low temperatures down to -12°C , after conditioning in a fog room or in air of 20°C and 80% RH. Resistances were measured at $+20^{\circ}\text{C}$, $+7^{\circ}\text{C}$, -3°C , -12°C .

Two cores of 50 mm diameter were taken for thin section analysis by polarising and fluorescent microscopy. Five cores were analysed for compressive strength and wet density.

3.2 Chloride profiles

Figure 3 shows chloride profiles obtained from the epoxy coated cycle-lane part and from the non-coated parapet part of the bridge.

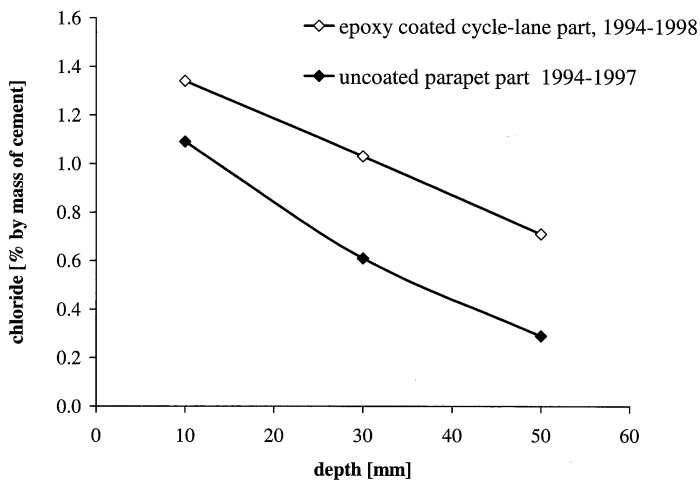


Fig. 3. Averaged chloride profiles, 1994–1998.

Figure 4 shows chloride profiles from different core samples, taken from the epoxy coated cycle-lane part of the bridge. The data were measured in 1994. It is clear that cores 1–6, 1–7 and 1–8 have similar chloride profiles. On the other hand core 1-12 has a much higher chloride contents between 20 and 60 mm depth.

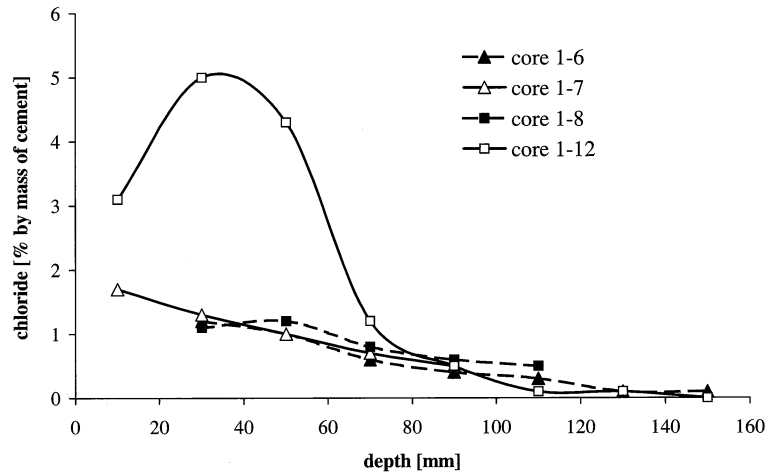


Fig. 4. Chloride profiles from the epoxy coated cycle-lane part of the bridge, data measured in 1994, core samples.

3.3 Resistivity results

While exposed in a fog room (20°C >95% RH) the resistivity of the concrete at 20°C measured on cores was about 200 Ω m; about 550 Ω m when exposed in a climate room at 20°C and 80% RH. These are normal values for rather dense Ordinary Portland cement concrete [COST 1997, Polder & Ketelaars 1991]. Exposed outside, the resistivity increased with decreasing temperature (420 Ω m at $T \cong 10^\circ\text{C}$). Measurements down to -12°C showed substantial increase of resistivity, from +20°C to -12°C by about a factor five, see Table 3.1. This trend is in agreement with other work on temperature dependence of concrete resistivity [Buerchler 1996, Bertolini & Polder 1997, Scotto et al. 1996].

Table 3.1. Resistivity of concrete from bridge deck in Ωm.

Temperature [°C]	Climate (storage before temperature change)	
	Fog room (20°C >95% RH)	20°C 80% RH
20	200	550
7	325	1000
-3	600	1500
-12	1250	2500

3.4 *Microscopy analysis*

For the investigation using polarising and fluorescence microscopy, samples were treated as follows (Larbi & Heijnen 1997). Blocks of 50 * 30 * 10 mm² were vacuum impregnated with epoxy resin containing a fluorescent dye. They were glued to glass plates and cut to 1 mm thickness. Then the samples were polished to about 30 µm thickness, allowing light transmission microscopy. With this technique, cement type, apparent water-to-cement ratio, voids microcracking and chemical reactions (ASR, dissolution) can be observed.

The results of thin section analysis can be summarised as follows. The concrete has been made with coarse Portland cement, most likely ordinary Portland cement equivalent to Dutch standard cement NEN 3550 Class A (CEM I 32.5), equivalent to ASTM Type 1. The aggregate is mainly quartz, quartzite and sericitic sandstone. The *w/c* - ratio of 0.45 – 0.55 is relatively low. The cement paste shows a moderate to poor homogeneity. The amount of air-voids (entrapped air) and compaction pores (excluding capillary and gel pores) was found to be about 3 % by volume of the concrete, which is reasonable for normal concrete. In general the microcrack density and microcracking patterns were found to be normal. Some alkali-silica gel in air-voids was found, but it has not caused significant cracking.

3.5 *Other results*

Carbonation was 2 mm or less in the top side, 5 mm or less in the underside. The carbonation depth of the bottom side is somewhat higher than on the top side, which is quite normal for horizontal slabs where CO₂ penetration is hindered by a higher moisture content of the concrete or the presence of a coating.

The mean compressive strength (of 5 cores) obtained in the laboratory was 60.5 MPa with a standard deviation of 9.1 MPa. The wet density of the concrete was between 2370 and 2400 kg/m³, which is normal for gravel concrete.

4 **Diffusion calculations**

The apparent chloride diffusion coefficient and the surface chloride content have been calculated by fitting the error function solution of the one-dimensional diffusion equation through the measured chloride profiles. The exposure conditions are not fully in agreement with this model, which presupposes water saturation. For this reason the diffusion coefficient is called an apparent diffusion coefficient. The results are presented in the next section.

5 Discussion

5.1 Overview of diffusion results

Tables 5.1 and 5.2 give an overview of all diffusion results for cycle-lane and parapets part, respectively. Figure 5 shows the diffusion coefficients (coated and non-coated parts). Figure 6 shows the chloride surface contents (coated and non-coated parts).

Table 5.1. Average and standard deviation of the apparent chloride diffusion coefficients and surface contents from the epoxy coated cycle-lane part of the bridge.

year	D_{av} [10^{-12} m ² /s]	D_{stdev} [10^{-12} m ² /s]	Cs_{av} [%]	Cs_{stdev} [%]	$n_{Profiles}$	remarks
1994	3.01	1.19	2.79	1.26	8	core samples A
1996	2.90	0.71	1.24	0.26	3	dust samples B
1998	0.86	0.38	1.19	0.25	6	dust samples C
1998	1.58	0.60	1.31	0.19	3	core samples D
1998	4.01	2.06	1.43	0.29	3	core samples E
av.	2.26	0.76	1.74	0.59	$\Sigma = 23$	

Table 5.2. Average and standard deviation of the apparent chloride diffusion coefficients and surface contents from the non-coated parapet part of the bridge.

year	D_{av} [10^{-12} m ² /s]	D_{stdev} [10^{-12} m ² /s]	Cs_{av} [%]	Cs_{stdev} [%]	$n_{Profiles}$	remarks
1994	0.91	0.37	1.62	0.44	5	core samples F
1997	0.43	0.16	1.38	0.38	3	dust samples G
1996	0.75	0.30	1.83	0.40	4	core samples H
av.	0.70	0.28	1.61	0.41	$\Sigma = 12$	

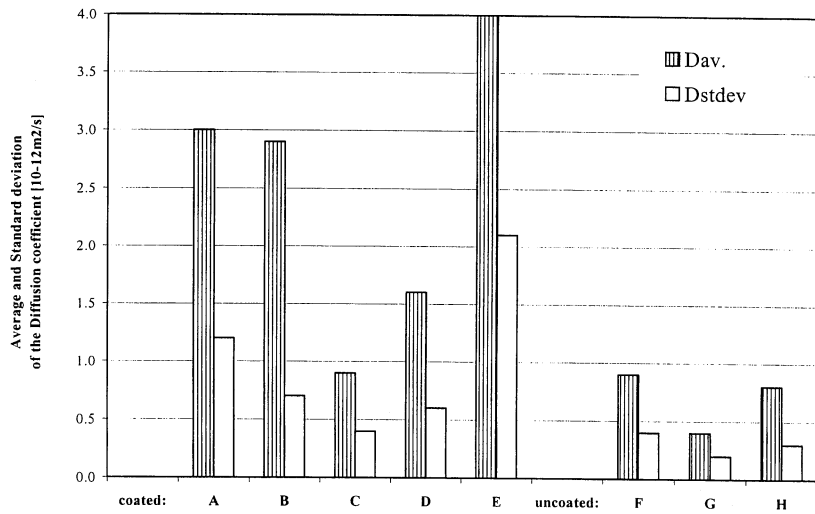


Fig. 5. Average and standard deviation of the apparent Chloride Diffusion Coefficient for the epoxy coated cycle-lane part and the non-coated parapet part of the bridge.

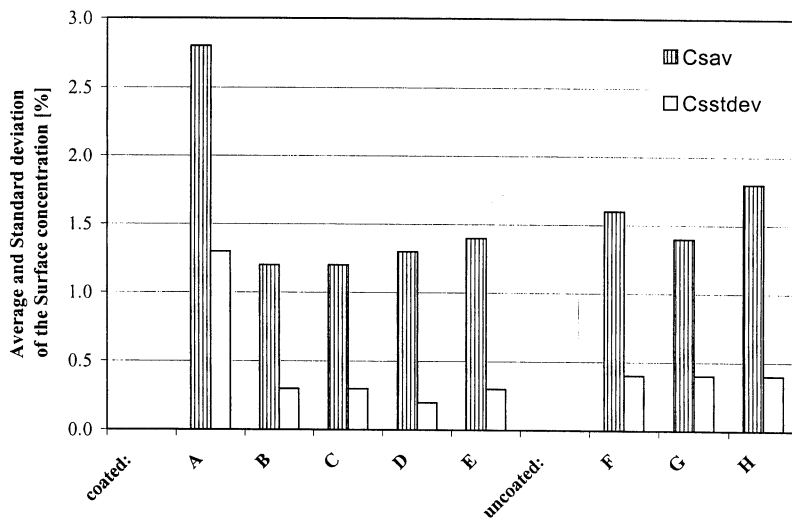


Fig. 6. Average and standard deviation of the surface content for the epoxy coated cycle-lane part and the non-coated parapet part of the bridge.

The results show a strong variation of apparent diffusion coefficients and surface contents, with both low and high values for D_{cl} and C_s . However further consideration can help to get a more clear picture.

5.2 *General considerations*

The concrete from the bridge may be supposed to have one single value of D_{cl} , which applies in reasonably saturated conditions and under a significant load of chloride. All samples contain significant chloride in the first layer; apparently de-icing salt loading has indeed occurred. If D_{cl} seems to be lower from calculations of some samples, then apparently the concrete was not saturated at those spots and transport has been slower than under saturated conditions. The overall D_{cl} is about $3 \cdot 10^{-12} \text{ m}^2/\text{s}$. This value is in good agreement with other data from Portland cement concrete, for instance after 16 years submersion in the North Sea [Polder & Larbi 1995]. This value corresponds well with the electrical resistivity according to the theoretical and empirical correlation [Andrade et al. 1994, Polder 1997].

5.3 *Cycle-lane*

The surface content C_s depends on the degree of protection given by the coating (in relation to cracks and defects in the coating) and the de-icing salt load. It may be supposed that the de-icing salt load has been homogeneous over the cycle-lane. Consequently differences in C_s in the cycle-lane have been caused by differences in perfection / state of damage of the coating.

So 2.8 % chloride by mass of cement (average; extremes up to 4–5%) is a reasonable worst case value for the surface content. A value like 1.2% (dust samples B, dust samples C) is a more general, average value (for less damaged coating, but still not perfect). Anyway the worst case determines the need for maintenance!

Realising that the coating has been effective for a period of time, some results can be reconsidered. In particular the profiles of data set "dust samples C" show an exceptionally low $D_{cl} = 0.86$. This value has resulted from a calculation based on an exposure period of 31 years. Now if at these spots the coating has prevented chloride ingress completely for say the first 15 years, the actual diffusion period is $31 - 15 = 16$ years. With this diffusion time, D_{cl} is about $1.7 \cdot 10^{-12}$, which fits better in the whole group of data. This suggests that at this spot, the coating has survived for considerable time. In other parts, it is likely that the coating has started to allow chloride penetration from 5 to 10 years age on.

5.4 *Parapet*

On the parapet, the de-icing salt load may be supposed to have been less than in the cycle-lane. C_s is rather homogeneous ($\sim 1.5 \pm 0.5$ % chloride by mass of cement). The average level of D_{cl} is quite low (for Portland cement concrete) with $0.7 \cdot 10^{-12} \text{ m}^2/\text{s}$. Such a low value could be viewed as to suggest that the concrete of the parapets is much denser than the concrete of the cycle-lane. This is obviously not realistic. Apparently (some of) the assumptions do not apply to the parapet, most likely that of water saturation. As the parapets were not coated, it is likely that the concrete has dried out considerably during dry periods. Subsequently, chloride transport has mainly taken place during and shortly after rain and snow periods. As a consequence a diffusion calculation is not correct and the

resulting D -values are not real diffusion coefficients. The observed values show considerable scatter (0.5 to $1.5 \cdot 10^{-12} \text{ m}^2/\text{s}$). This may be the result of differences in moisture content between individual spots on the parapet.

5.5 *Coating performance*

If over the years the coating becomes completely ineffective, C_s will approach 4-5 %. The rate of the loss of protection is unknown. However, it suggests that from a certain age on, the protection by the coating is lost over a progressively increasing part of the deck. It should be noted that after about 25–30 years so much chloride had reached the reinforcement, that at least 50% of the deck suffered severe delamination due to corrosion!

The degradation of the coating on this bridge has not been systematically investigated as such. This type of coating consists of a layer of coating proper and a layer of crushed stone particles, applied onto the still uncured material. The crushed stone improves the road surface properties. The mechanical loads exerted by the traffic will push the aggregate into the coating and gradually cause loss of integrity. Consequently it is not surprising that its performance deteriorates substantially over time [Rodrigues, Fedrizzi 2000].

It should also be realised that the concrete investigated here was not taken randomly from the old bridge deck. Firstly, during the demolition process, pieces were selected with no more than moderate visible damage; parts that fell apart too easily could not be transported. Secondly, from the pieces received parts were selected with minimal delamination, in particular for the cathodic protection tests. Summarising, the concrete tested here is probably representative for the “better” parts of the bridge with respect to protection offered by the coating. Consequently it is very well possible that the average service life of the coating is even shorter than suggested from the results discussed here.

5.6 *Service life*

In this section the **average** service life will be discussed because a full probabilistic analysis was not carried out.

Starting from the loss of protection by the coating in the cycle-lane, chloride diffusion has been fast ($3 \cdot 10^{-12} \text{ m}^2/\text{s}$) and the surface content has become high (C_s up to 4–5 %), so corrosion initiates after 5–10 years from the loss of integrity of the coating. Considering that at least 50 % of the deck showed severe delamination after 25 years, the loss of protection by the coating must have started at 10–15 years! This is illustrated in Figure 7, showing the calculated chloride profile after 29 years for the average diffusion coefficient and for the average plus and minus the standard deviation (with the surface content fixed at the average value). Figure 8 shows the chloride profile for the average surface content and average plus and minus the standard deviation (with the diffusion coefficient fixed at the average value).

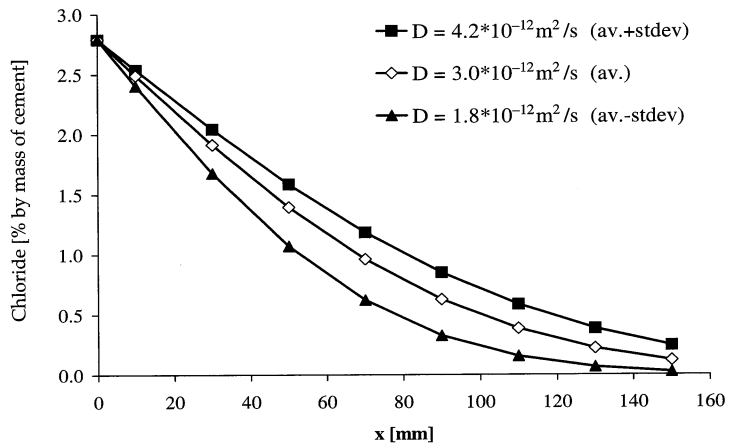


Fig. 7. Calculated chloride profiles for cycle-lane part (epoxy coated), exposure time = 29 year, $C_s = 2.8\%$.

For a typical cover depth of 30 mm (upper side of the deck), the chloride content would be about 2 % after 29 years (Figure 7), or between 1 % and 3 % (figure 8). Given the concrete composition, these chloride contents most likely would initiate corrosion.

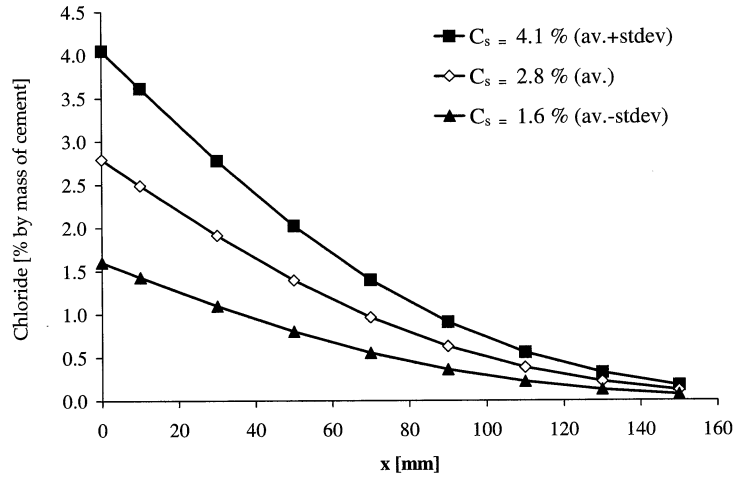


Fig. 8. Calculated chloride profiles for cycle-lane part (epoxy coated), exposure time = 29 year, $D = 3 \cdot 10^{-12} \text{ m}^2/\text{s}$.

A similar approach shows a relatively mild chloride profile in the parapets. Calculated chloride profiles after 29 years are shown in Figures 9 and 10 for the average and extreme diffusion coefficients and surface contents. Generally after 29 years the chloride content at rebar depth (30 mm) is 0.6 – 0.8 %. If we extrapolate to longer service lives, the calculations show that on the parapet, even the worst case ($D_{cl} = 1.3 \cdot 10^{-12} \text{ m}^2/\text{s}$, $C_s = 1.6 \%$) probably will not cause corrosion initiation before an age of about 50 years.

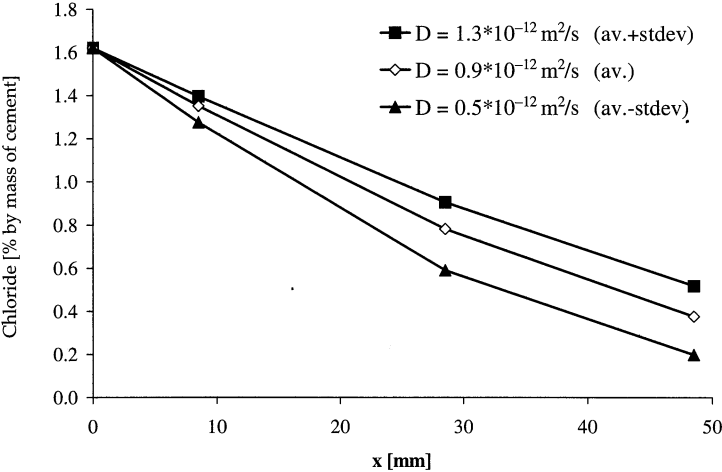


Fig. 9. Calculated chloride profiles for parapet part (uncoated), exposure time = 29 year, $C_s = 1.6 \%$.

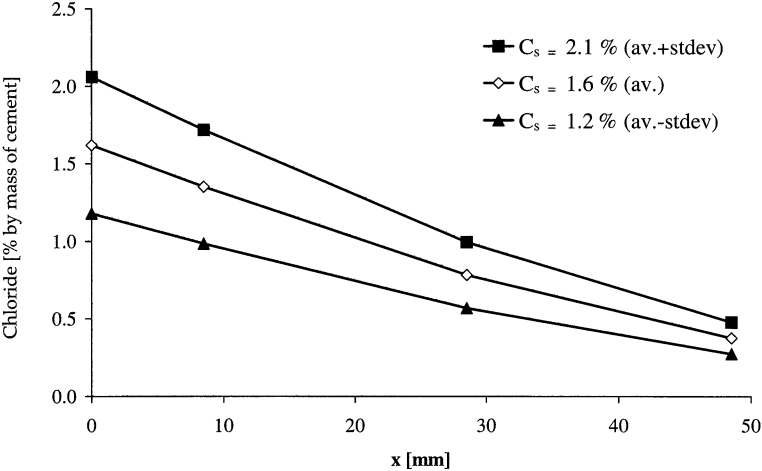


Fig. 10: Calculated chloride profiles for parapet parts (uncoated), exposure time = 29 year, $D = 0.9 \cdot 10^{-12} \text{ m}^2/\text{s}$.

How realistic this analysis for the parapets is, is unknown. As discussed above, the diffusion calculation is an (over)-simplification for the parapets. The calculation for the parapets presented here, however, is supported by the observed absence of significant corrosion damage in the parapets of all slabs that were obtained after the bridge was demolished. For these simplified service life calculations it has been assumed that cracks in the concrete had no effect on chloride transport.

The service life of the bridge deck must be considered separately for the cycle-lane and the parapet. On the cycle-lane, de-icing salts are applied directly. A coating was applied as a protective barrier against this threat. The damage to the deck after about 30 years shows that the coating has not been effective on the long run. The calculation results give some further insight. It appears that the integrity of the coating was lost after some 10 to 15 years on a general scale. Chloride penetration has started before that time on a local scale. After penetration started, chloride transport has been fast because the concrete was relatively wet. This may have been an effect of the coating: Cracks in the coating allow water to penetrate during rain and which spreads throughout the concrete. Subsequently, the coating slows down evaporation in dry periods, even if cracks are present in the coating.

On the parapet, the chloride load has been less, but not zero. If the concrete would have been wet, transport would have been faster and corrosion might have initiated. The fact that no significant damage was present in the parapets, is probably caused by the relatively dry state of the concrete.

Comparing the relatively favourable state of the parapets with the poor state of the cycle deck, the effect of the coating applied to must be reconsidered. It appears that the coating may have had a protective effect for some years, but after deterioration of the coating had started, the effect on the durability of the deck has been negative!

A few suggestions for practice can be inferred:

- If a coating is applied as a barrier against de-icing salt penetration, the integrity of the coating must be monitored (e.g. by resistivity sensors detecting water ingress). The first signals of water ingress should be a signal for maintenance of the coating.
- Before the application of a protective coating, the concrete should be hydrophobised to prevent water absorption in case the coating deteriorates; hydrophobic treatment of concrete has a long service life, in particular if degradation by ultra violet light is prohibited by the coating [Polder et al. 1999].
- The owner should realise that protective coatings have a limited service life. The coating on this deck should probably have been replaced or at least should have received serious maintenance every 10 years.
- There appears to be a need for coatings which prevent water (and salt) ingress, allow evaporation and resist mechanical loads to which a cycle bridge is subjected.
- Alternatively, for this type of structures the cover to the reinforcement should be increased and/or a cement type with a high resistance to chloride penetration should be chosen, such as blast furnace slag cement with >65% slag (CEM III/B).

7 Conclusions

The apparent chloride diffusion coefficient of the concrete from the investigated bridge in wet conditions is typical for Portland cement concrete, about $3 \cdot 10^{-12} \text{ m}^2/\text{s}$.

In the cycle-lane part of the bridge, this diffusion coefficient and chloride surface contents of about 2–4 % describe the penetration of chloride rather accurately, despite the presence of an epoxy coating. It appears that the protective effect of the coating was practically lost after about 10 to 15 years of service on a general scale. The presence of the damaged coating has caused the concrete to remain relatively wet, allowing quick chloride transport to deeper parts.

The non-coated parapets on the other hand, have not shown quick chloride penetration despite significant chloride loads. The calculations give quite low apparent diffusion coefficients for the parapet, which are not in agreement with the assumption of (near) water saturation (and the typical values for Portland cement concrete). Apparently the absence of a coating has allowed the concrete of the parapets to dry out substantially, which has slowed down chloride transport.

The protective effects of (epoxy) coatings in these types of structures should be reconsidered. It is suggested to apply hydrophobic treatment to the concrete before applying this type of coatings and / or to monitor coating integrity. There appears to be a need for improved coatings for this type of applications, ideally combining high chloride penetration resistance, high water evaporation, good mechanical resistance and a long service life.

8 Acknowledgements

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