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Case studies of concrete deterioration in a marine environment in Portugal

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Abstract

The long-term behaviour of concrete structures has shown that their main cause of distress is reinforcement corrosion. One of the most aggressive exposure conditions for concrete is the marine environment. In these conditions chloride penetration and chloride-induced reinforcement corrosion rates can be very high, often leading to a reduced service life. This paper describes a series of case studies of different types of concrete structures, subject to several exposure conditions in the marine environment, that suffered extensive deterioration due to chloride-induced corrosion. Several degrees of deterioration associated with different exposure conditions are described and different types of corrosion both in reinforced and prestressed concrete structures are illustrated.

Keywords: Reinforcement corrosion; Concrete structures; Marine environment; Chlorides; Repairs; Docks; Wharves; Bridges

1. Introduction

Reinforced concrete has been in the last decades one of the most used building materials. It has proved to be a reliable structural material with very good durability performance when properly used. However, there are many structures which show early deterioration, namely those exposed to aggressive environments.

Experience shows that the corrosion of the reinforcement is the main cause of structural concrete deterioration. This type of damage is responsible for the huge financial costs spent each year on the repair of deteriorated structures.

The main reasons for the poor performance of concrete structures are poor workmanship and lack of knowledge of the deterioration mechanisms which result in insufficient planning and wrong estimation of environmental effects. Nowadays, it is well known that the durability of materials and structures depends both on the environmental conditions at the exposed surfaces of the structures and on the material resistance to the action of aggressive substances. The understanding of these two main aspects is the key for the design and execution of durable structures and for the rational approach to the repair of damaged structures.

This paper presents some case studies where structures exposed to the marine environment showed early deterioration due to reinforcement corrosion. In these case studies the causes of the poor performance of the structures referred to above are well illustrated. Several degrees of deterioration associated with different exposure conditions are described and steel corrosion both in reinforced and prestressed concrete structures is illustrated. Repair strategies for extending the service-life of those structures are also presented in this paper.

2. Case studies

2.1. Docks

This example describes the deterioration of the docks of a shipyard located in the estuary of River Sado, in the west coast of Portugal. The shipyard has three reinforced concrete docks (docks 20, 21 and 22) built during the 1973–1975 period. Dock 20 is a ship construction platform with plan dimensions of 420×75 m and 8 m high walls. Dock 21 is intended for ship repair and its dimensions in plan are 450×75 m with 18 m high walls. Dock 22 is also intended for ship repair and its dimensions in plan are 350×55 m with 12 m high walls. Fig. 1 shows the cross-sections of the three docks.

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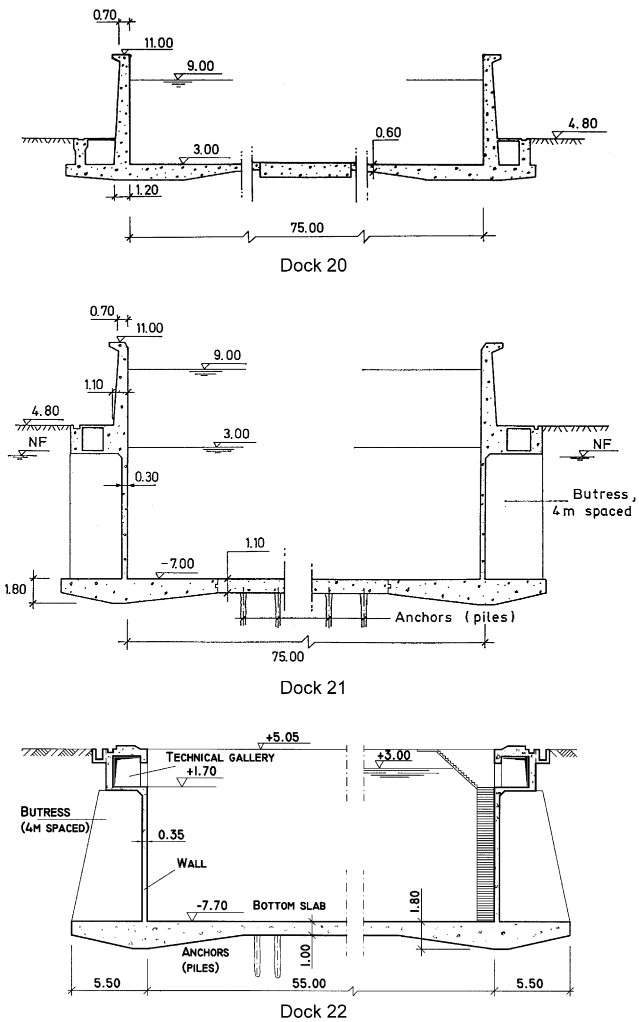

Fig. 1. Cross-section of the docks; all measurements in m.

The docks were built with a poor-quality concrete. The specified strength class was B225 corresponding to a characteristic cube strength of 22.5 MPa. No durability criterion was considered. Documentary information about the structure indicated that the concrete mix contained a cement content of 300 kg/m^3 and the water/cement ratio was 0.7. The concrete cover specified for the structure was 4 cm for the walls and 6 cm for the bottom slabs. The placing, consolidation and curing of concrete were inadequate, showing some segregation zones and poor concrete joints. Quality control was poor.

In 1991 an extensive number of tests were performed with a view to characterise the concrete of the docks. Regarding the compressive strength a characteristic value of 17.6 MPa was obtained for the walls and 20.6 MPa for the bottom slab. This means that the specified strength has not even been achieved. The concrete of the walls showed a water permeability coefficient of 14×10^{-11} m/s and the concrete of the bottom slab exhibited a value of 0.6×10^{-11} m/s. The open porosity (porosity accessible to water) measured on the concrete of the walls and slab was 17.8% and 16.4%, respectively.

The docks are exposed to a marine environment characteristic of a temperate climate. The water composition of the estuary varies year-round and is influenced by the flow rate of the river Sado. The measurement of the main aggressive elements of the water during the 1 year period has given the following results:

Chlorides, Cl^-	16–21 [g/l]
Sulphates, SO_4^{--}	2.1–2.9 [g/l]
Magnesium, Mg^{++}	1.1–1.6 [g/l]
pH	7.8–8

Docks 21 and 22 are submitted to fairly frequent filling, i.e., they are usually filled twice a month. Dock 20 is filled once or twice a year.

In 1978 the docks, after 4–5 years of service, already showed signs of deterioration with cracking and spalling due to steel corrosion, mainly in the internal walls of dock 20. This abnormal behaviour led to some studies which clearly showed the phenomena of chloride-induced reinforcement corrosion due to a poor concrete quality and the ingress of chlorides from the contact water. The carbonation depth was much lower than the concrete cover. Tests performed along the dock walls showed that the carbonation depth ranged from about 10 to 25 mm.

The deterioration process developed along the years extending to several parts of the docks. The deterioration rates depended mainly on the exposure conditions of each zone. The worst situation was observed on the internal side of the walls of dock 20. After 5 years of exposure, the concrete was deeply contaminated by chlorides at the reinforcement level as shown in Fig. 2. After 10 years of exposure, a general delamination and spalling of the cover concrete occurred, up to the level reached by the salt water when the dock was filled as shown in Fig. 3.

After the beginning of corrosion, the resistivity of the concrete is, together with the access of oxygen to the reinforcement, one of the main factors determining the corrosion rate. The measured values of resistivity ranged from 4 to 8 kΩ cm on the walls. Although the interpretation of resistivity measurements is empirical, these low values are generally associated with high corrosion rates [1,2].

It is interesting to note that the loss of cross-section of the reinforcement was not high in these walls. The main effect of the corrosion process was the spalling of the concrete cover. In these walls general corrosion took place and macro corrosion cells, generally associated with chloride attack and very high corrosion rates, were not observed.

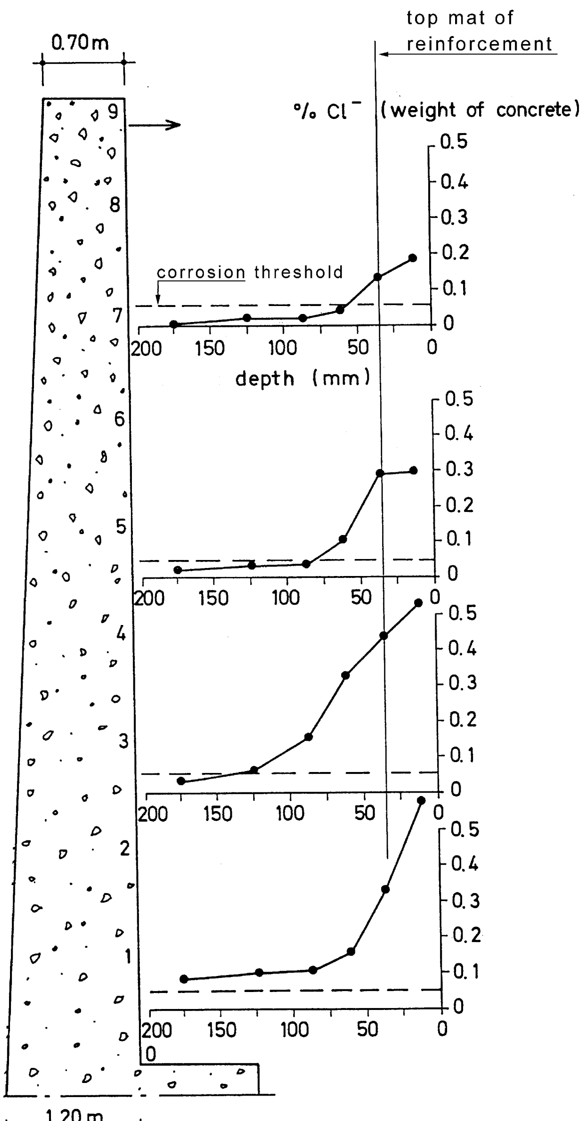

Fig. 2. Chloride profiles measured on the internal surface of dock 20 walls (1980); measurements in m.

In dock 20, the bottom slab and the external face of the walls also showed a high deterioration level although not so severe as in the internal walls.

The extremely high deterioration rate observed in the internal walls can be explained by the exposure conditions and the poor concrete quality. Regarding the ingress of chlorides into concrete, different physical mechanisms may be distinguished: permeation, capillary absorption and diffusion. Permeation and absorption are very fast transport mechanisms while diffusion is much slower. As a result of the wet and long dry cycles of the internal walls of the dock and the high capillary porosity of the concrete, the absorption process was the dominant transport mechanism. So, after the first cycles of dock filling the reinforcement was depassivated. The high chloride concentration, the low resistivity of the concrete, and the easy access of oxygen to the reinforcement during the dry periods led to a high corrosion rate which caused the early spalling of the concrete cover.

In dock 21, two main degrees of deterioration were observed on the internal surfaces of the walls. The upper part of the walls showed a general spalling of the concrete cover, similar to that observed in dock 20. The exposure conditions of those zones are the same, since the upper part of dock 21 walls is in contact with salt water only when dock 20 is filled (dock 20 is connected to the estuary through dock 21). The lower part of the walls showed much less deterioration. The different deterioration conditions exhibited by the zones of the walls previously mentioned is due to the exposure conditions of each zone which affect both the chloride ingress and the corrosion rate.

At the lower zone, the wall is subject to frequent wetting cycles (about 48 h twice a month). The concrete at that area usually presents a high saturation level caused by this high filling rate and by the permeation of water from the reverse side of the wall. Under these conditions the dominant transport mechanism of chlorides is diffusion. Fig. 4 presents the chloride profiles in the lower part of the walls measured after 16 years of exposure, showing high concentration but much less than that observed in dock 20 after 5 years of exposure.

In dock 22, a similar deterioration process was observed on the internal surfaces of the walls. Due to the saturation of the concrete in the lower parts of the walls, the corrosion rate is controlled by the cathodic process due to the limited access of oxygen to the reinforcement. This explains the much less concrete spalling in these zones. It has also been observed that the northern wall of the dock 22 showed a deterioration level higher than that on the east and west walls, as shown in Fig. 5. This fact is also due to the exposure conditions. Exposure to the sun of the northern wall is fairly higher than that of the other walls. Therefore, it is exposed to higher temperatures, which lead to a quicker drying of the concrete and to a higher access of oxygen to the reinforcement. It is also known that the temperature increases the corrosion rate significantly. Thus, these two factors together led to a higher deterioration level of that wall.

Macro corrosion cells, i.e., corrosion cells with well-separated anodes and cathodes, were observed on these walls, as shown in Fig. 6. This phenomenon is characteristic of chloride-induced corrosion and is due to the high conductivity of the concrete caused by water and chlorides in the pore structure. The measured values of concrete resistivity ranged usually from 1.5 to 3.0 kΩ cm. The anodic zones in these macrocells increased in time, giving rise to the type of deterioration observed in Fig. 5. This process can lead to very high

Fig. 3. Typical deterioration of dock 20 walls after 10 years of exposure.

corrosion rates when the cathodic areas are much larger than the anodic ones. The corrosion rate was measured through the loss of section of the bars in several anodic zones of the walls. Values of 200–400 μm/year were frequently obtained and in some cases values in excess of 600 μm/year were measured. These high values are similar to those obtained in several studies [3–6] where the corrosion rates were measured both in situ and in laboratory tests on elements exposed to chloride-contaminated environments.

In dock 21, an accident occurred due to a failure of a structural element, deeply affected by reinforcement corrosion, as shown in Fig. 7. A wall of a technical gallery collapsed under water pressure when the dock was completely filled and a ship was manoeuvring. This almost caused the emptying of the dock and severe damage to the ship. The collapse of the wall was mainly caused by the loss of bond between the bars and concrete due to the splitting of the concrete cover. This accident shows the significance of reinforcement corrosion on structural safety.

The bottom slabs of docks 21 and 22 showed less deterioration than that of the walls. The survey of zones with delamination of concrete cover showed that their area represented about 10% of the total area of the slabs. The lower deterioration rate observed in these elements results from the higher saturation level of the concrete and to the accumulation of dirt and water on the surface which helps restrict access of oxygen to the reinforcement.

To assess the deterioration rate of the bottom slabs, measurements of the corrosion rate were performed using the polarisation resistance method [6]. The measured values range from 17 to 23 μm/year. Considering that the exposure conditions of the bottom slabs do not vary significantly along the year one can assume that the values of the corrosion rate will not vary significantly either. The concrete resistivity measurements showed very low values for this parameter, usually below 1.5 kΩ cm. These low values of the concrete resistivity and corrosion rate indicate that the corrosion rate is being controlled by the cathodic process (see for instance [7]).

The repair of the docks has been carried out in several stages along the last 16 years according to the development of the deterioration process and financial possibilities of the owner. In 1984, 3600 m^2 of the internal face of the walls of dock 20 were repaired. In 1988, 5600 m^2 of the upper part of the internal face of the walls of dock 21 were also repaired. The other part of the walls of docks 20 and 21, internal and external faces, were repaired in 1991, amounting to a total area of 23,400 m^2. The repair of the walls of dock 22 was carried out in 1999 involving an area of 14,200 m^2. Until now there has been no intervention in the bottom slabs due to financial constraints, although the bottom slab of dock 20 presently shows more than 50% of the total area spalled or delaminated, requiring urgent repair.

The repair methodology adopted consisted of the total removal of the superficial layer of deteriorated and chloride-contaminated concrete to a depth beyond the reinforcing bars, 2 cm minimum, and replacement by a new high-quality concrete. The basic principle of repair associated to this type of intervention consists in the repassivation of the reinforcement, stopping the anodic process.

Although another methodology could have been used for the repair of the docks, i.e., the application of a

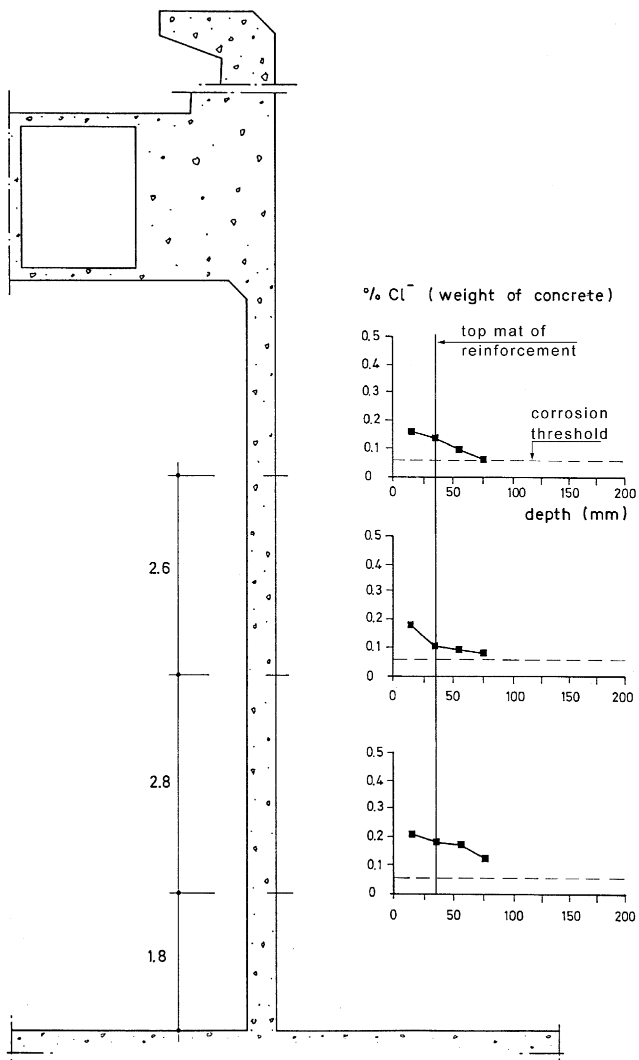

Fig. 4. Chloride profiles measured on the lower part of dock 21 walls (1991).

cathodic protection system, the repair technique was adopted for being considered to be more cost-effective in these cases.

The monitoring of the repair works carried out in 1991, which consisted of measuring chloride penetration into concrete over a 5 year period, proved that this repair technique is reliable. The prediction of long-term chloride penetration into dock 21 walls showed that no significant problems should be expected within a period of about 20 years [8]. This was the service-life required by the owner.

In the last repair works performed in dock 22, embedded sensors for monitoring the corrosion process were used, as shown in Fig. 8. Several types of sensors were installed: reference electrodes to monitor the electrochemical potential of reinforcement, macrocell sensors to monitor the onset of corrosion, and temperature and resistivity sensors. The location of the macrocell sensors at several depths in the concrete cover allows the measurement of the penetration of the critical chloride concentration.

2.2. Wharves

This case refers to the deterioration of four wharves located at the same shipyard complex as described in Section 2.1. The length of the wharves varies between 120 and 200 m and their width is 20 m. The deck of the wharves consists of six precast prestressed concrete beams parallel to the length and a 30 cm thick cast-in-place concrete slab, prestressed in the transverse direction. The beams are supported by hollow piles founded on the seabed.

The wharves were built during 1973–1975. The concrete strength grade was B300 for the cast on site slab

Fig. 5. Deterioration of the northern wall of dock 22 (1999).

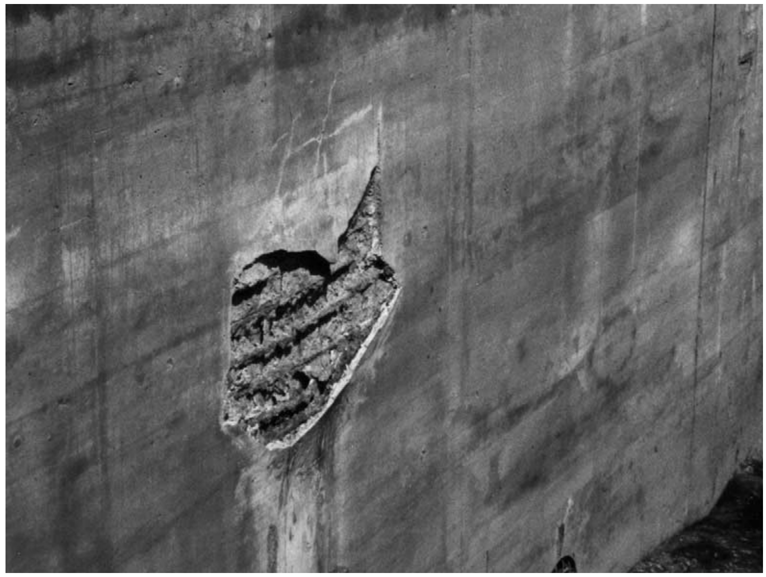

Fig. 6. Macro corrosion cell in dock 22 walls.

Fig. 7. Failure of the gallery wall in dock 21.

and B350 for the precast beams, corresponding to a characteristic concrete cube strength of 30 and 35 MPa, respectively. No durability criterion was specified. The reinforcement concrete cover is about 3 cm for the normal steel and 5–8 cm for the prestressing steel. The present Portuguese standards specify a minimum concrete cover of 4.5 cm for ordinary reinforcement and 7 cm for prestressed reinforcement.

An in-depth inspection, carried out to assess the condition of the wharves, revealed several deterioration degrees related to the different structural elements and exposure conditions.

The wharves are subject mainly to three types of marine exposure: the deck slab and the upper part of the beams are exposed to the splash and spray zone; the lower part of the beams and the upper part of the piles are exposed to the tidal zone, while the lower part of the piles are subject to the submerged zone of the marine environment.

The tests performed during the in-depth inspection showed that the concrete was heavily contaminated by chlorides to a depth far beyond the reinforcement in almost all the structural elements of the wharves, as shown in Fig. 9.

In the beams the chloride content at the level of the prestressed tendons is generally in excess of 0.1% by weight of concrete, while in the slabs it is lower (less than 0.05%) except in a critical zone below the crane rails, where the cover of the tendons is about 5 cm due to the execution of gutters in the slab to put in the rails.

The electrical resistivity of the concrete measured on the slabs and upper parts of the beams (elements

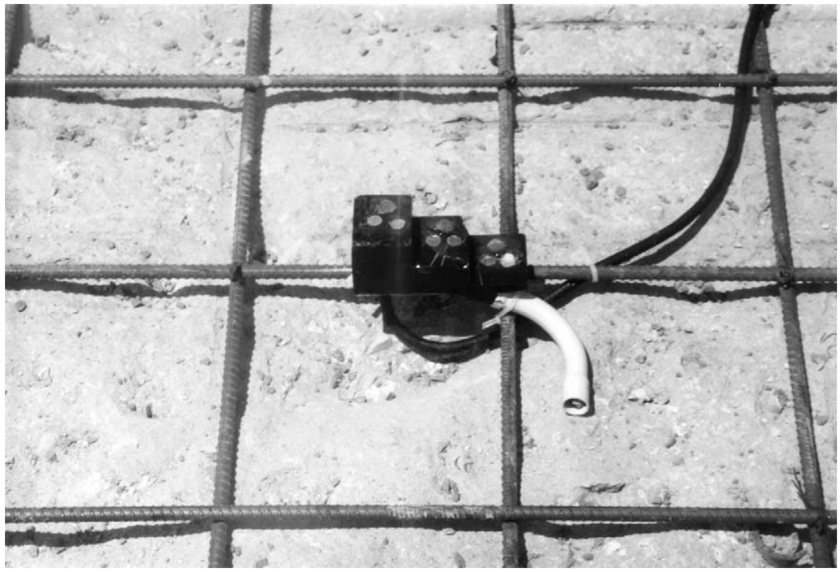

Fig. 8. Sensors for monitoring the corrosion process.

exposed to the splash and spray zones) was typically between 5 and 15 kΩ cm while the lower parts of the beams and the piles (exposed to the tidal zone) presented values below 5 kΩ cm.

Potential mapping surveys using a silver/silver chloride electrode indicated that there was a high probability of corrosion of the reinforcement in all the test zones (potentials lower than −250 mV were measured in all zones) [9,10]. In the tidal zone very low potential values (−450 to −550 mV) were measured on several tests areas. These low values are reported to be associated to situations in which the corrosion rate is limited by the access of oxygen to reinforcement [10,11].

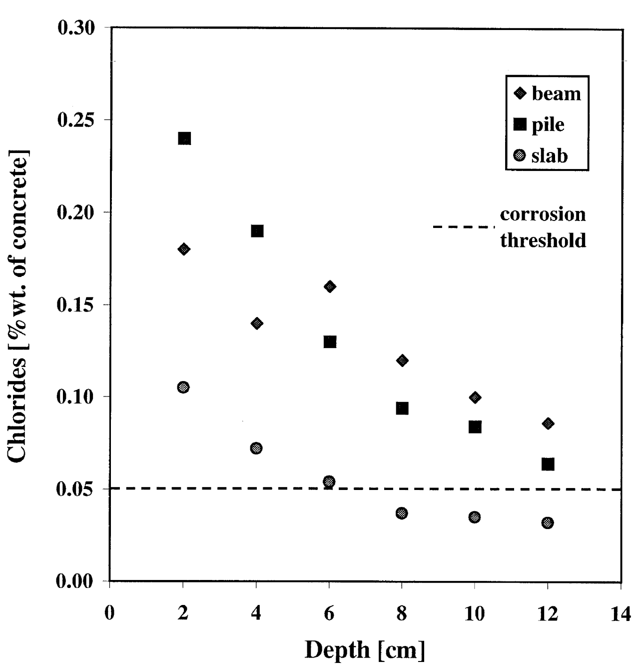

Fig. 9. Typical chloride profiles in different structural elements of the wharves.

Two main different degrees of deterioration were observed in the wharves. The upper part exposed to the splash and spray zones presented large areas with spalled and delaminated concrete due to reinforcement corrosion. The lower part exposed to the tidal zone presented a much lower degree of deterioration. In this zone only small delaminated areas were observed. Fig. 10 illustrates the difference of the deterioration conditions between the splash and tidal zones.

A substantial loss of the cross-section of the bars was observed in the deck of the wharves. In some zones, 25 mm diameter bars were totally corroded after 24 years of exposure, as shown in Fig. 11. This accounts for a corrosion rate in excess of 500 μm/year, assuming uniform corrosion around the bars. The extremely high corrosion rates developed in the deck bars can be explained by the macrocell chloride-induced corrosion mechanism with large cathodic areas and small anodic zones as discussed before.

In some places of the wharves exposed to the upper part of the tidal zone (near the high-tide level) the development of black or green corrosion was detected. This type of corrosion usually occurs in places where the water content of concrete is very high, limiting the access of oxygen to the anodic zones. The corrosion products do not give rise to a large increase in volume and may diffuse into the pore structure of the concrete without causing delamination. This corrosion process is very serious since it can lead to high losses of the reinforcement section without any visible warning.

Fig. 10. Typical deterioration observed in the splash and tidal zones.

In one of the wharves a very critical deterioration case was detected. The prestressed tendons of the deck slab located below the gutters that contain the crane rails showed a deep corroded condition. The steel ducts were almost all destroyed by corrosion and in many cases it seems that stress corrosion cracking of the wires had occurred, as shown in Fig. 12. This type of corrosion causes the brittle failure of the prestressing steel without significant loss of cross-section. Sudden failure of the structural elements may occur since there is no visible warning of tendons deterioration. Several cases of bridge collapse due to this type of corrosion process are reported [12].

In some cases, ducts were left totally un-grouted due to poor workmanship. In these situations the prestressed wires are not protected by the alkaline environment of the grout and a drastic degradation condition was found, as shown in Fig. 13.

Fig. 11. Bars with 25 mm diameter totally corroded.

The extreme deterioration condition of the prestressing tendons was caused by a very aggressive microenvironment engendered in the rail gutters associated to poor materials and workmanship. The lack of drainage of the gutters led to the accumulation of salt water during long periods on the concrete surface. The water evaporation leaves behind the salts which lead to growing chloride concentrations. These conditions caused a high penetration rate of chlorides into concrete and led to the early development of the corrosion mechanism on the tendons.

Several tests were performed to assess the prestressing steel condition of the beams. Radiography photographs were used to detect voids in the prestressing ducts localised in the web of the external beams. Windows were opened on the web faces of the external beams to expose the tendons, allowing the direct observation of the condition of the ducts and the removal of grout samples from the inside of the ducts for chloride analysis. Although the concrete around the tendons is contaminated by chlorides, no significant corrosion signs of the ducts were observed. This can be explained by the saturation of the concrete around the ducts since the tendons are located in the lower part of the beams which is exposed to the tidal zone. No chloride contamination was detected in the grout of the tendons.

The repair methodology adopted for the wharves was in general the same as used for the dock except in the case of the deck slab with deeply deteriorated tendons. Here it was necessary to strengthen this structural element. The strengthening consists of a 20 cm thick cast-in-place high-quality concrete overlay (water/cement ratio = 0.30; cement content = 400 kg/m^3; microsilica

Fig. 12. Stress corrosion cracking of the prestressed wires.

content = 40 kg/m^3) and ordinary reinforcement bars. Due to financial reasons, there was no intervention in the beams. A cathodic protection system should be implemented in the near future to protect the beams and other structural elements exposed to the tidal zone.

2.3. Bridge

This example refers to the deterioration of a 35 year old reinforced concrete arch bridge. The arch bridge located near the seaward has a span of 278.40 m and two approach viaducts, the total length of the bridge being 493.20 m.

The deck slab is 25.0 m wide and has 12 beams supported by four columns with spans of 21.20 m. Over the river the columns transfer their loads to the two twin arches, 8 m wide each, with a double caisson cross-section.

The bridge is exposed to an atmospheric marine environment and the concrete is never directly in contact with seawater. The central part of the bridge (arch bridge) is more exposed to the wind, and therefore to the salt from blown spray and salt-laden mist.

A very good quality concrete was achieved in the construction. The measured characteristic compressive cube strength was 50 MPa, and the mix composition presented a water/cement ratio of 0.32. The measured

Fig. 13. Corrosion of prestressing steel in un-grouted ducts.

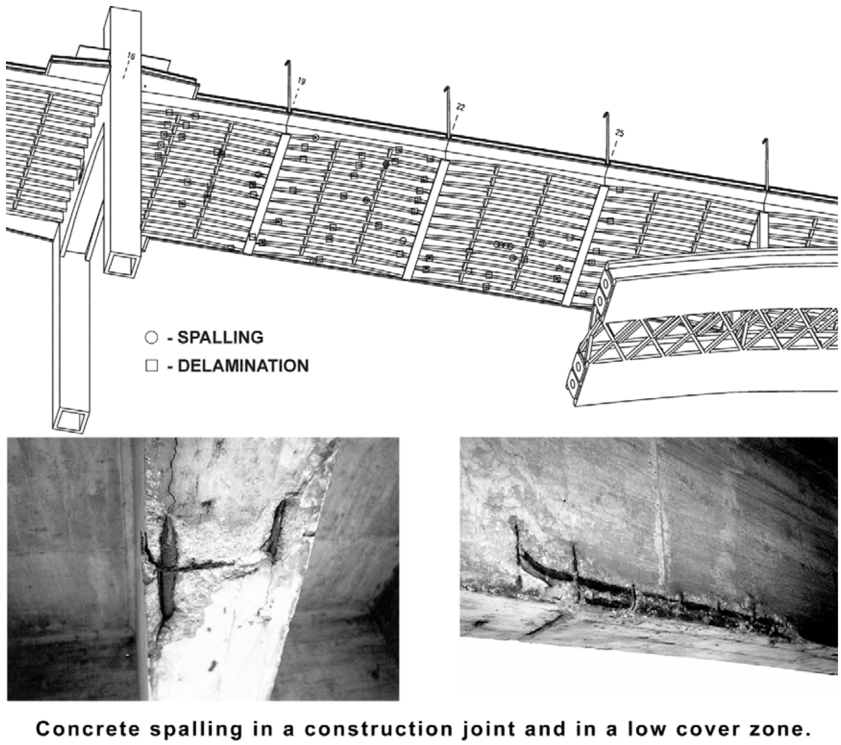

Fig. 14. Typical deterioration observed along the deck.

capillary absorption was about 0.05 $mm/\sqrt{min}$ and the water permeability about 10^{-12} m/s. The concrete surfaces of the bridge were painted. This constitutes an additional protective measure against the ingress of aggressive substances.

No relevant maintenance was done in 35 years and the road authority decided to perform an in-depth investigation to assess the structural conditions of the bridge.

The main find of the investigation was significant delamination and spalling of concrete occurring in the deck beams mainly over the arch. Fig. 14 shows the recorded information of the typical concrete deterioration observed in the inspection of the deck. Higher incidence of deterioration was observed in the downstream faces and lower corners of the beams.

The main cause of the concrete deterioration of the bridge was poor execution quality since the concrete quality was very good. Poorly made construction joints and honeycombing were frequently observed and the thickness of concrete cover presented a high scatter. The inspection showed that there was a significant area of the beams with a concrete cover thickness less than 2 cm.

Carbonation measurements did not exceed 5 mm depth, even in the areas of deficient concrete placing. Chloride penetration was in general small except in areas with defects where values up to 0.06% by weight of concrete were observed at 3.5 cm depth, as summarised in Fig. 15. Chloride-induced corrosion was thus the cause of deterioration in these areas.

High values of the electrical resistivity of the concrete were measured on the deck beams. In general values in excess of 40 kΩ cm were obtained and only in a few zones were values around 30 kΩ cm measured. The

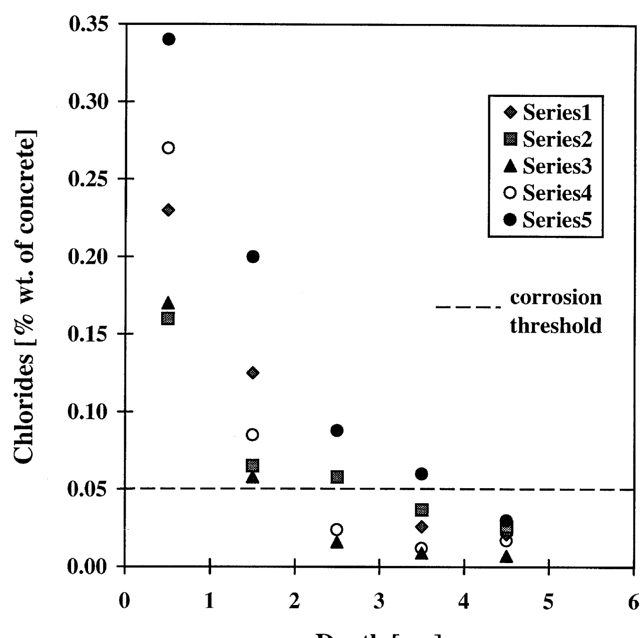

Fig. 15. Chloride profiles in zones with defects.

corrosion rate measurements using the polarisation resistance method show that on the areas without defects the reinforcement is in a passive condition (values below 1 μm/year were measured). On a poorly made construction joint a value of 7 μm/year was measured. Experimental studies [13] show that a bar cross-section loss of only 20 μm is sufficient to crack the concrete cover, and a crack width of 0.3–0.4 mm is reached when the radius loss of the bar is about 100 μm. This may explain the delamination and spalling observed on the defective zones of the deck beams.

The loss of cross-section of the reinforcement observed on the spalled zones was low. This can be explained by the high values of concrete resistivity which prevent the formation of macro corrosion cells. The corrosion rate is controlled by the electrolytic process.

The repair strategy adopted for this bridge was local patch repair with cement mortar followed by an application of a protective coating. The basic principles behind this repair methodology are the following: stopping the anodic process where the reinforcement is corroding; application of a surface barrier to control the ingress of further aggressive substances and also to limit the moisture content of the concrete. This last aim of the surface protection is very important since it is impossible in practice to assure that all the contaminated concrete is removed in a patch repair methodology. A surface protection impermeable to water and permeable to water vapour allows concrete to dry out raising its resistivity and controlling in this way the electrolytic conduction process of the corrosion mechanism.

3. Conclusions

The assessment of the structures referred to in this paper shows that, when defective structures are exposed to aggressive environments, such as the marine environment, very high deterioration rates can be developed leading to serious damage conditions in very short time periods.

The inspections carried out showed that the principal mechanism responsible for the extensive deterioration of the structures studied is chloride-induced reinforcement corrosion. This mechanism led to extensive delamination and spalling of the reinforcement concrete cover.

The deterioration rate observed in these defective structures depends mainly on the exposure conditions of each part of the structure. The higher deterioration rates were observed in zones where the concrete surface is subject to wetting and drying cycles of salt water. In these very aggressive exposure conditions, corrosion rates in excess of 500 μm/year were measured.

When prestressing steel is used, supplementary protection measures must be implemented because of the high sensitivity to corrosion of this type of steel, as shown by the deterioration of the tendons in a deck slab of a wharf.

The repair costs, both financial and environmental, of the presented structures are very high. To avoid these unacceptable situations, planning and execution must be based on sound design and good-quality workmanship.

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