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# PROGNOSTIC EVALUATION OF MANAGEMENT NEEDS FOR TWO BRIDGE STRUCTURES

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## ABSTRACT

Corrosion of steel reinforcement in bridge structures is of concern to the State Road Authorities (RAS) who control in excess of 8,000 concrete bridges and 16,000 culverts. This paper deals with investigations on two Victorian bridges, located in an aggressive marine environment, to estimate their service life and determine their management needs. The bridges are multi-span structures carrying major traffic over tidal rivers and are 9 and 30 years old. The work relates to the identification of potential corrosion problems at an early stage, before damage becomes visible, so that early intervention can be adopted to protect the integrity of the structure and extend their service life.

Both field and laboratory investigations have been carried out on the bridges, including electrochemical measurements (half-cell potential, resistivity, corrosion rate), strength testing, determination of carbonation depth and chloride penetration profile, and petrography examination of concrete.

The results have shown that the concrete quality is good and there is currently no visible corrosion. However, considerable $Cl^-$ has penetrated the concrete at the base of columns and pile cap, but it has not reached the level of the reinforcement. Electrochemical data also indicate that these areas are at risk.

Based on the results of the investigations the paper discusses the remaining service life of the structure and its management needs in order to extend its service life.

**Keywords:** concrete, marine environment, corrosion, chloride diffusion, corrosion rate

## 1. INTRODUCTION

In Australia, the State Road Authorities (RAS) control in excess of 8,000 concrete bridges and 16,000 culverts, a large number of which are located in the coastal areas and are subjected to aggressive environments and at risk of corrosion of steel reinforcement. Many more bridges (in excess of 20,000) are under the management of Local Governments, some of which are in similar situations. Corrosion of steel reinforcement in bridge structures is of concern to the SRAs, and condition surveys are periodically conducted to detect deteriorating structures and to monitor them for initiating appropriate intervention.

In a recent unpublished report, (RC7007AA-April 1999), and on the basis of condition surveys conducted by various consultants, the present authors estimated that about 20-25% of the bridges for which survey reports were available, could be considered at risk of corrosion and

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needing immediate attention. For the other bridges service lives of 35-100 years were predicted on the basis of analysis of carbonation depth and/or chloride ingress into the cover concrete. A main shortfall in the methodologies applied was that service life was not predicted on the basis of corrosion rate of the reinforcement, but on the basis of parameters which are indirectly related to it. The measurement of corrosion rate was suggested to be included in the methodology, and three bridges were investigated for this purpose.

This paper provides data on two Victorian Bridge structures around Melbourne, about 9 and 30 years old. They were nominated for investigation so that potential problems could be identified and rectified, or appropriate preventative action implemented before visible damage occurs.

## 2. THE BRIDGE STRUCTURES

The two bridges investigated are:

- Lynch's Bridge over Maribyrnong River (Melbourne bound structure) on Smithfield Road.
- Footscray Road Bridge over Moon Ponds Creek (Melbourne bound).

### *2.1. Lynches Bridge*

The section of the bridge investigated (Melbourne bound) was built in 1991, and has five spans, each 21 m, and six piers. Pier 1 and 6 function as abutments standing on dry land and piers 2 to 5 are in the tidal river. Pier 2, which was more accessible, was considered a representative area and was selected for the investigation. Construction documents give the strength grade of concrete at 50 Map, and cover to reinforcement 70 mm for pile caps and columns, and 45 mm for cross heads.

Visual inspection showed that the bridge was in good condition. Mud and moss covered the pile cap, but no obvious deterioration could be seen after removing the dirt from a section. The base of pier columns is subject to tidal seawater, to a height of about 300-mm, which is covered by green moss. Corrosion stains (apparently unrelated to reinforcement) and horizontal cracks appear at the lower part of the column where the surface color changes to muddy apparently marking the maximum tide level (this may be the construction joint, according to the construction drawings). There is no other obvious deterioration, but predicting future corrosion activity in pier columns and pile caps was of interest to VicRoads.

### *2.2. Footscray Road Bridge*

This bridge (Melbourne bound section) was constructed in 1962 and has five spans (12.8 m each), and four wall piers between the abutments. The deck consists of an array of prestressed concrete beams and an asphalt overlay. The width of bridge (Keri-to-Keri) is 16 m. The design concrete strength was characteristic cylinder strength at 28 days of 39.3 Map. Three of the piers sit in tidal seawater, and only at high tide water rises against the base of Pier 1 (particularly the west side) so it is subject to water level fluctuation.

Visual inspection showed that the concrete of the main structural members is generally in good condition. Deterioration was found on the edge of the deck overhang, which shows reinforcement corrosion and concrete sapling. Some longitudinal hairline cracks were seen at the underside of some beams. Occasional fine vertical cracks were also seen in the pier walls, which are of no serious concern. Drawings show the presence of a mesh of $\phi$ 6 mm steel bars on the faces of the pier walls at 70 mm depth. Pier 1 and the deck beams in this area were selected for the investigation.

## 3. INVESTIGATION METHODOLOGY

The bridges was subjected to both field and laboratory investigations. The field investigation included:

- visual inspection and selection of measurement sites
- determination of depth of reinforcement (cover thickness) in selected areas
- determination of half-cell potential in the selected areas
- determination of electrical resistance of concrete in the selected areas
- Determination of corrosion rate of reinforcement in representative areas.

The laboratory work was conducted on cores (100 mm diameter) extracted from representative areas of the bridge. Drill holes were reinstated using Render HB40, a polymer modified cement mortar and a bonding agent and surface treatment compound called Nitobond AR. The laboratory work included determination of:

- carbonation depth
- $CL^-$ content ( profile) and estimation of $CL^-$ diffusion coefficient
- compressive strength of concrete
- water absorption, density and volume of permeable voids (VPV)
- Petrography features of concrete.

## 4. RESULTS AND DISCUSSION

### *4.1. Half-Cell Potential, Resistively and Corrosion Rate*

Corrosion Activity Criteria

Half-cell potential and concrete resistively results were obtained using a Great Dane model GD3000 with an Erg/agile electrode, and the data converted to $Cu/CuSO_4$ electrode potentials, which are summarized in Figure 1. The criteria for half-cell potential in ASTM C876, relating potential values to the likelihood of corrosion activity, are as follows:

More negative than –350 mV CSE – active corrosion is likely
Between –350 to –200 mV CSE – corrosion activity is uncertain
More positive than –200 mV CSE – corrosion activity is unlikely

The results of resistively measurements could be empirically interpreted as:

> 1100-kΩ·cm – The resistively will effectively stop corrosion
50 - 100 kΩ·cm – Low corrosion rate
10 - 50 kΩ·cm – Moderate to high corrosion rate when steel is active
< 10-kΩ·cm – Resistively is not the controlling factor

Many factors such as moisture and salt contents, carbonation of concrete, cement composition, etc. influence the resistively of concrete and should be taken into account in the interpretation of results.

The corrosion rate of reinforcement measured by instruments without a guard ring has been interpreted (Clear, 1989) as follows:

| | | |
|---|---|---|
| No corrosion expected | - | $I_{con} < 0.2\ \mu A/cm^2$ |
| Corrosion possible in 10 to 15 years | - | $I_{con} = 0.2$ to $1.0\ \mu A/cm^2$ |
| Corrosion possible in 2 to 10 years | - | $I_{con} = 1.0$ to $10\ \mu A/cm^2$ |
| Corrosion possible in 2 years or less | - | $I_{con} > 10\ \mu A/cm^2$ |

In our work, the corrosion rate of reinforcement was determined at locations of interest using the Decor 6 linear polarization device with censored guard ring to confine the area of the reinforcement bar for which the current density is measured. Field and laboratory work have demonstrated that the corrosion current density ($I_{con}$) determined by this instrument can be categorized as follows (Broomfield [1] and Gu et al [3]):

| Corrosion status of reinforcement | Value of $I_{con}$ ($\mu A/cm^2$) |
|---|---|
| Passive condition | < 0.1 |
| Low to moderate | 0.1 – 0.5 |
| Moderate to high | 0.5 – 1.0 |
| High | > 1.0 |

These values are almost an order of magnitude lower than those for unconfined measurements.

*4.2. Measurements on Bridges*

Half-cell potential maps of both column 4 and column 5 in Lynches bridge show that below a height of about 850-mm (Y4 on the graph) the potential is lower than –350 mV CSE. This is the criterion for likelihood of active corrosion, as recommended by ASTM C 876, and becomes even more negative towards the base of the columns. Correspondingly, the concrete resistance is low, below 5-kΩ·cm, and is considered conducive to corrosion. These indicate a high risk of reinforcement corrosion at lower parts of the columns, although the very negative half-cell potential (-700 mV CSE) at the base areas of the columns may partly be attributed to the water saturation that induces oxygen starvation. However, direct measurement of corrosion rate at the base of column 5 shows a corrosion current density of 0.216 $\mu A/cm^2$, which is considered to be a low–to–moderate corrosion rate.

In higher areas, the half-cell potential ranges over -150 to -350 mV CSE, where corrosion activity is uncertain according to ASTM C876. The resistively is between 10 to 20 kΩ·cm in this area, which is considered to facilitate a moderate corrosion rate when steel is active. The measured corrosion rate for heights of 1.0 and 1.5m above the column base were 0.027 and 0.012 $\mu A/cm^2$, respectively which indicate negligible corrosion activity. Therefore, the interpretation of resistively and half-cell potential values by the ASTM categories was not reliable.

For the Footscray Road Bridge, half-cell potential and resistively maps were obtained for the whole of the west face of Pier 1 and for the soft of beam 1. Figure 1 shows the map for only zone

5 in the middle area of the pier, where the corrosion rate measurements were also done. The half-cell potential of the pier wall below a height of 0.35 m from the ground is more negative than –300 mV CSE, and that 0.35 m to 0.70 m high is between –200 and -300 mV CSE. The concrete resistively in this area is generally in the range 0 to 20-kΩ cm. These results indicate possible risk of reinforcement corrosion in the lowest 0.35 m of the pier wall. The upper parts have higher potentials ( -100 mV) and higher resistively values, and are Considered safe.

Direct measurement of corrosion rate at the middle part of the pier wall shows a corrosion current density of 0.12 μA/cm$^2$ to 0.35 μA/cm$^2$ from the top to base. This demonstrates a low to moderate corrosion rate, and indicates that the corrosion was probably related to chloride ingress, which is expected to be larger at the base. The values of corrosion rate, in this case, appear to be in agreement with the half-cell potential trend down the pier wall.

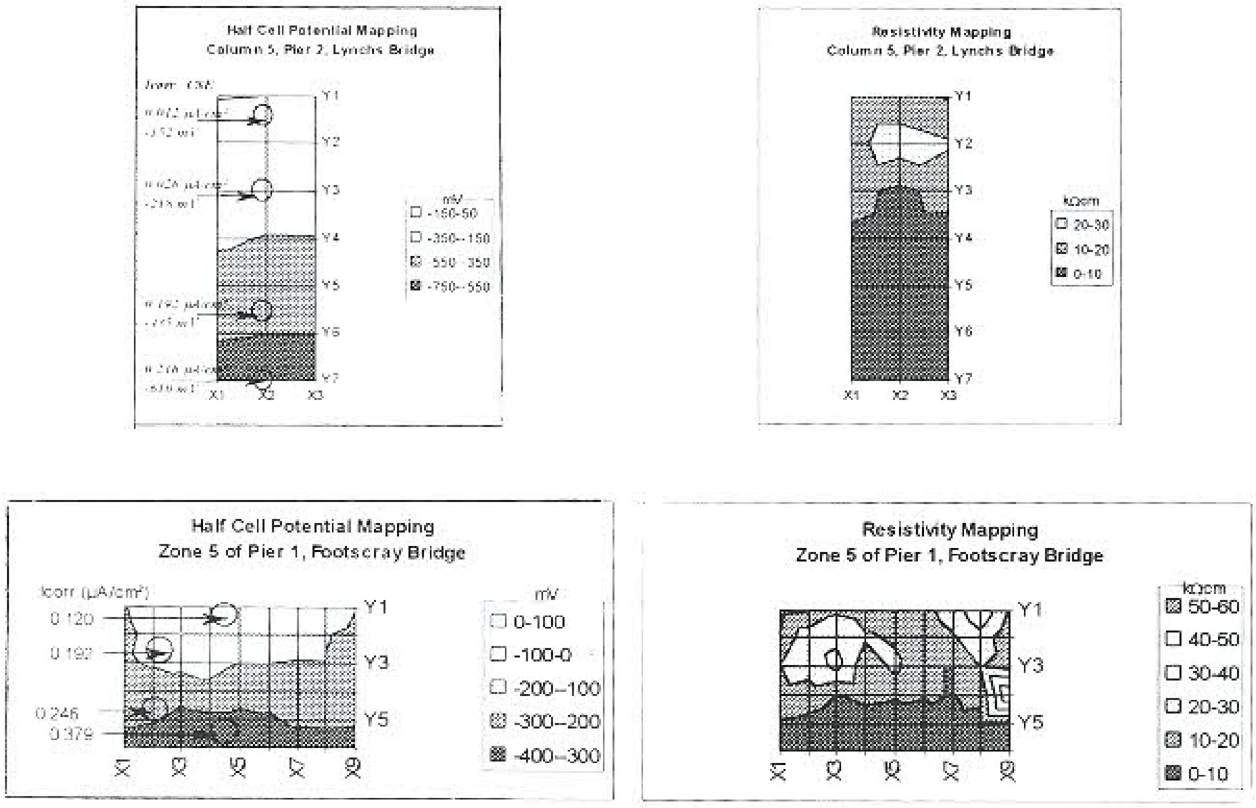


Figure 1 Half-cell potential and resistively maps for the elements indicated in the two bridges

## 5. LABORATORY INVESTIGATIONS

*5.1. General properties*

Table 1 lists the core samples taken from various bridges and their location in the structures.

Table 1. List of cores, their ID and location

| Cores ID | Location | Cover depth (mm) |
|---|---|---|
| Lynn 1 | Pile cap | 135 – 160 |
| Lynn 2 | Pile cap | 144 |
| Lynn 3 | Pier 2, column 5, at waterline | 70 – 100 |
| Lynn 4 | Pier 2, column 5, 1.5 m above Pile cap | 70 – 100 |
| Lynn 5 | Pier 2, column 4, at waterline | |
| FTCL-1 | Pier 1, water line, Melbourne side, 3.7 m from North side | 70 |
| FTCL-2 | Pier 1, near the top, 2 m from North side | 74 |
| FTCL-3 | Pier 1, water line, 7.4 m from North side | 70 |
| FTCL-4 | Pier 1, mid height, 1 m from top, 9.4 m from North side | 73 |

Table 2 provides a summary of the general properties of the concrete cores. The results show that cores from each of Lynches Bridge and Footscray Road Bridge have relatively uniform and adequate properties, but that the latter seems to have a lower strength grade concrete. However, the density, water absorption and permeable voids volume are comparable in the two concertos, probably indicating similar durability characteristics.

Table 2. Results of various laboratory tests on the cores

| Core sample | Carbonation depth (mm) | Water absorption (%) | Permeable voids volume (%) | Dry Density kg/m$^3$ | Compressive strength Map |
|---|---|---|---|---|---|
| Lynn 1 | < 1 | 6.7 | 15.7 | 2248 | - |
| Lynn 2 | < 1 | - | - | - | - |
| Lynn 3 | < 1 | - | - | - | 54 |
| Lynn 4 | 5.5 | 6.7 | 15.6 | 2263 | 63.5 |
| Lynn 5 | 2.0 | 6.6 | 15.3 | 2235 | 71.5 |
| FTCL-1 | 3.6 | 6.7 | 15.9 | 2328 | 42.0 |
| FTCL-2 | 10.2 | 6.6 | 15.8 | 2282 | 39.8 |
| FTCL-3 | 1.5 | 6.5 | 15.6 | 2322 | 37.3 |
| FTCL-4 | 9.8 | 6.0 | 15.1 | 2327 | 41.2 |

## 6. Chloride Penetration

Figure 2 shows the chloride penetration profiles for the various elements tested in the two bridges. The curves are fitted according to FICA's second diffusion law to the experimental data. Table 3 contains the fitted values of surface chloride content and the diffusion coefficient.

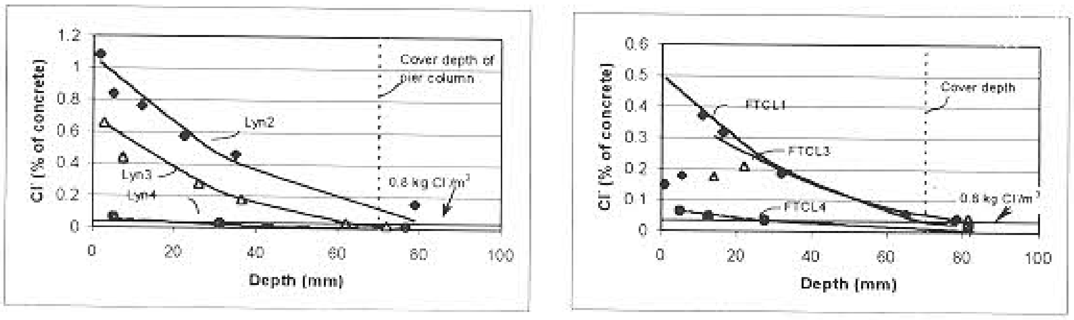


Figure 2 Chloride penetration profiles for the various elements

Table 3. Fitted values of surface chloride content and the diffusion coefficient

| Cores ID | Location | $C_s$ (%)* | D ($m^2/s$)# |
|---|---|---|---|
| Lynn 2 | Pile cap of pier 2 | 1.07 | 2.85E -12 |
| Lynn 3 | Pier 2, column 5, at waterline | 0.70 | 1.84E -12 |
| Lynn 4 | Pier 2, column 5, 1.5 m above Pile cap | 0.07 | 2.13E -12 |
| FTCL-1 | Pier 1, water line, 3.7 m from North side | 0.50 | 6.36E -13 |
| FTCL-3 | Pier 1, water line, 7.4 m from North side | 0.40 | 9.02E -13 |
| FTCL-4 | Pier 1, 0.7 m above ground, 9.4 m from North side | 0.07 | 5.39E -13 |
| * Surface $CL^-$ content (%) by concrete mass. # $Cl^-$ diffusion coefficient. | | | |

For Lynches Bridge, the concrete at the water line or below tidal zone has a high chloride content, despite the fact that a coat of paint protects it. This must have occurred as a result of the exposure of the concrete to salt water at a very early age. The surface chloride content of these concrete samples is very high, being nearly 1% by mass of concrete, indicating the bridge pier is in a very corrosive environment. The $CL^-$ diffusion coefficient is about $2 \times 10^{-12}$ $m^2/s$ which is normal, however at the high end, for structural concrete.

Chloride content at the reinforcement level of pier columns is approaching the threshold limit. It can be calculated that at the waterline, $CL^-$ at 70 mm depth (design cover thickness) will reach the 0.8 $kg/m^3$ limit (about 0.035% by concrete mass) in two years. In fact the measured cover thickness was in the range of 75 – 100 mm, which would give a longer service life. For higher chloride threshold values a somewhat longer time would be expected, e.g. 4 years and 11 years for threshold levels of 0.05% and 0.10%, respectively. Considering most steel bars in piles / columns are subject to chloride diffusion from both sides of the corner, i.e. two-dimensional diffusion, the actual time would be somewhat shorter. It is recommended to take the estimation for the lower threshold, (i.e. 0.035% or 0.05%) for columns. In the higher up areas, where the surface $Cl^-$ is only marginally higher than the threshold, there would not be any risk of chloride-induced corrosion initiation in 70 years. No chloride determination was made for the area between 0.3 to 0.8 m above the pile cap, but it can be reasoned that the service life would be in between those predicted for the zones above and below it, depending on height above water line.

The $Cl^-$ content of Footscray Bridge pier concrete at the water line is also high, reaching the 0.8 kg/m$^3$ level (0.035% by concrete mass) at the steel mesh. However, if corrosion takes place at higher threshold values, then it would take 4 years to reach the 0.05% level and 33 years the 0.10% level. In the higher up location, the 0.035% threshold content occurs at about 30 mm and 0.05% at 12 mm. Calculations based on the estimated diffusion coefficient, and the maximum surface $CL^-$ build up reached at the water line, show that it would take more than 70 years for $CL^-$ at steel level to reach the 0.035% threshold value. This is a very conservative estimate because the surface $Cl^-$ at higher parts of the pier may never reach that at the water line.

The carbonation depth of the concrete at the higher location is 10 mm, considering that the age of the bridge is 37 years, it can be concluded that the carbonation front would not reach the steel (70 mm) in foreseeable time. Therefore, there is no risk of carbonation-induced corrosion damage during the service life of this structure (100 years).

## 7. SERVICE LIFE OF CONCRETE ELEMENTS

Based on the results of the measurements, the following observations are made:

*7.1. Lynches Bridge*

The summary of service life prediction for this bridge is given in Table 4. At this stage, no definitive evaluation can be given of the risk of corrosion-induced cracking because, to do this, it would be necessary to calibrate Decor and establish a correlation between the corrosion density measured by the Decor and actual corrosion mass loss. It has been reported that Decor gave comparatively lower corrosion currents, e.g. the data presented by (Weyers[5]) indicates that Decor reading is approximately 20% of that calculated from the actual mass loss. If this is confirmed then the pier column base of Lynches Bridge would have an actual corrosion current of around 1 μA/cm$^2$. The current density of 1 μA/cm$^2$ is equivalent to 11.6 μm steel loss / year. Given the same corrosion current density (which in fact would vary with time) a section loss of 0.58 mm would result after 50 years. The structural consequence of this section loss needs to be computed.

Assuming a threefold increase in the volume of rust compared to the steel it replaced, an expansion of about 35 μm / year would result. The build up of expansion stresses would eventually result in cracking of the cover concrete. This situation would apply if adequate oxygen supply allowed rust formation and accumulation at the steel surface. (Rodriguez et al 1996), applying $I_{corr}$ =100 μA/cm$^2$ for 100-200 days, produced general corrosion of 0.3-0.6 μm and pitting corrosion of 1-5 mm in steel bars. They found a significant reduction in the elastic modulus, and a 40% decrease in the bearing capacity of the columns tested.

It could be expected that 1 μA/cm$^2$, for the base of the column in this bridge, applied for a period of 30-50 years would probably produce similar effects to that of the higher current applied by Rodriguez et al [4] for the shorter duration. Based on the results of (Weyers [5]) and taking into account the 32 mm diameter of reinforcement bars in Lynches bridge compared to the 16 mm bars mentioned in this publication, it can be suggested that corrosion induced cracking may occur in about 3 years after chloride content at the reinforcing bar exceeds the threshold, which will take at least 2 years. Thus, in the worst case, some cracking may be expected to occur after 5 years.

The risk of corrosion-induced cracking is low in the water-saturated concrete, e.g. pile cap and the low-waterline area of column base, because the corrosion products will remain in ionic form due to lack of oxygen. However, section loss may arise from the dissolution of iron, and the dissolved iron may diffuse out and result in concrete surface staining by rust, and this should be monitored. If the degree of steel loss is the same (11.6 μm / year) as mentioned above, the consequence of 0.58 mm section loss on the load bearing capacity of the column bases would need to be evaluated. This loss may result in debunking of steel from concrete, and asymmetric deterioration of the steel bars and ligatures, which could cause buckling of the column under extreme conditions. Pitting corrosion, which could penetrate much deeper into the steel, may aggravate the deterioration and could cause earlier failure than expected.

Table 4. Summary of service life prediction for Lynches Bridge

| Location | Threshold CL | Potential | | Corrosion rate | | Comments | Service life |
|---|---|---|---|---|---|---|---|
| | To reachre-bar (year) | CSE (mV) | Corrosion activity | $I_{corr}$ (μA/cm²) | Category | | years to repair |
| Pile cap | 0 | - | - | - | - | Under water. Rust stain may occur, but rust build up and cracking is unlikely due to lack of oxygen. Steel section loss and debunking is likely in long-term. | Zero according to CL level, i.e. intervention needed. Catholic protection recommended. |
| Column water line | 2 | -550 – -750 | active | 0.216 | low - moderate | As above | 2 years (according to CL level). As above – early intervention option. |
| Column 0.3-0.8 m | - | -350 – -550 | active | 0.192 | low – moderate | Cracking likely 5 years* | Monitor further |
| Column higher up | 50 | -150 – -350 | uncertain | 0.012 | passive | No risk in 50 years | 50 years at least |

* In the worst case, 2 years for further CL penetration, and 3 years for rust to develop.

The concrete at the water line region (tidal zone) appears to need the most urgent attention. Because of chloride contamination of the pile cap concrete, catholic protection may be required. The higher part of the columns, e.g. between 0.3 to 0.8 m, where half-cell potential is between -350 to -550 mV, has a very low rate of corrosion (0.027 $\mu A/cm^2$), and has little risk of developing cracking. Above 0.8 m the corrosion rate is 0.012 $\mu A/cm^2$ with even less corrosion activity.

*7.2. Footscray Road Bridge*

The piers in this bridge are constructed with a steel mesh (made up of 6-mm bars) at 70 mm below the concrete surface, and light reinforcement elsewhere. It is not possible to predict the time to corrosion-induced cracking, as there is no corrosion-cracking data published for this size of steel bar. It can be reasoned that the small bar size and large cover thickness make cracking unlikely to occur in the near future

When the corrosion rate is low-to-moderate. However, more research work is needed to estimate the cracking potential, and give definitive conclusions. With regard to the piers that are submerged under water, similar arguments could be presented with respect to possible section loss as for the submerged sections of columns in Lynches bridge. Table 5 summarizes the information for Footscray Bridge.

Table 5. Summary of service life prediction for Footscray Bridge Pier

| Location | Threshold Cl. to reachre-bar (year) | Potential | | Corrosion rate | | Comments | Service life years to repair |
|---|---|---|---|---|---|---|---|
| | | CSE (mV) | Corrosion activity | $I_{corr}$ ($\mu A/cm^2$) | Category | | |
| Water line | 0 | -400 – -300 | active | 0.379 | low - moderate | Uncertain on cracking * | 0 Monitor |
| 0.3 – 0.6 m high | - | -300 – -200 | uncertain | 0.246 | low - moderate | Cracking unlikely | 70 |
| 0.7 m high | 70 | -300 – -200 | uncertain | - | | Cracking unlikely | >70 |
| 1 m and above | - | -200 – -100 | passive | 0.192 – 0.120 | low - moderate | Cracking unlikely | >70 |

* Model for estimation needs to be developed. A small bar diameter of 6 mm and relatively large cover thickness of 70 mm probably makes the occurrence of cracking unlikely.

## 8. RECOMMENDATIONS

The data obtained on Lynches bridge indicated that monitoring the development of corrosion is necessary for this bridge. The high chloride level in the concrete is the major cause for concern. In this case, and if monitoring indicates enhancement of corrosion activity, the best available technique would be catholic protection.

Coating of the concrete by paint at the column base may not be effective in reducing further corrosion, because the chloride level at reinforcement has already reached the threshold for corrosion initiation. Chloride removal may not be an option in this case, due to constant contact with salt water. Catholic protection would require regular maintenance and ongoing monitoring. The choice of the catholic protection technique and types of anode, would depend on the budget, and would be decided by VicRoads and the contractor. The higher parts of the piers would need maintaining the integrity of the present surface coating and regular general maintenance.

Footscray Road Bridge is generally in good condition. The sides of the parapet walls, which have shown some localized corrosion and concrete sapling need to be patch repaired, care being needed to avoid enhancing the corrosion in other areas by the patch repair. No deterioration is evident on the beams, and corrosion current was very small. However, because of the presence of a few hairline cracks further monitoring of the beams is recommended.

The situation with the base of the pier walls is the same as that of the column bases of Lynches bridge. Considering the age of this bridge and the fact that steel corrosion has been initiated, catholic protection should be considered for the pier walls, but further monitoring is essential in establishing the need for such intervention.

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