REHABILITATION AND MONITORING OF SAWTELLS INLET BRIDGE – 12 YEARS LATER

Fred Andrews-Phaedonos, GeoPave, VicRoads
Ahmad Shayan, ARRB Transport Research Pty Ltd
Aimin Xu, ARRB Transport Research Pty Ltd

SYNOPSIS

The Sawtells Inlet Bridge was the subject of a major rehabilitation undertaken in three stages in 1991 and 1992. This comprised the replacement of superstructure components, construction of a strengthening and waterproofing deck overlay, conventional patch repairs of parts of the substructure and application of suitable protective coatings for the whole of the bridge, in both atmospheric and tidal/splash zone microclimates. A permanent corrosion monitoring system in the form of half-cell potential reference electrodes and resistivity pins was also installed at the time, to enable the ongoing monitoring of the effectiveness of the various concrete repair methods and protective coatings in limiting chloride induced corrosion of concrete coastal bridges in Victoria. Additional testing over the years included chloride and pH testing, and more recently corrosion rate measurements, as well as visual inspection and deamination survey. Recent investigations and assessments reveal that although after nearly 12 years of service a significant proportion of the concrete repairs and protective coatings are approaching the need for re-intervention in this very aggressive marine environment, overall the combination of some repairs and associated coatings has proven to have performed in a reasonably satisfactory manner. Analysis of selected elements has shown that sufficient amounts of chloride ions have penetrated to the level of steel reinforcement in the columns, although these commenced from an already high base prior to the application of the protective coatings. This has been associated with low to moderate corrosion rate of the steel, as indicated by the half-cell potential values and the direct measurement of corrosion rate of the steel, particularly in the unprotected control areas and the lower portions of the columns. Prestressed concrete deck planks that had been installed as part of the rehabilitation process showed that the extent of chloride ingress and corrosion was insignificant. This is due to the dual protective coating (silane and anti-carbonation coating) applied to the planks after installation in the bridge.

As a result of this ongoing monitoring over the 12 year period, it may be concluded that both a polymer modified cementitious coating and an epoxy coating have been relatively effective in limiting the ingress of chlorides into the concrete piers, despite their shortcomings which may be attributed to mixing, application, curing and other operational practices. The application of polymer modified cementitious repair material does not seem to have caused or enhanced macrocell corrosion in the neighbouring coated areas. Because of significant chloride levels in the columns, consideration may have to be given to the installation of a CP system (vs. ongoing patch repairs and coatings) to protect the piers from further progress of corrosion, based on a life cycle costing analysis.

ACKNOWLEDGEMENTS: The authors wish to thank the Chief Executive of VicRoads, Mr David Anderson for permission to publish this paper. The views of this paper are those of the authors and do not necessarily represent those of VicRoads or the ARRB.

1. INTRODUCTION

Sawtells Inlet Bridge was constructed in 1968 across the Tooradin Channel within the township of Tooradin, approximately 61 km southeast of Melbourne. The bridge is located 0.5 km from the coastline and is subjected to tidal movements of more than 2.5 m. The substructure becomes submerged up to the mid-level of the pier crossheads. This is a 3-span U-slab bridge simply supported on RC abutments and two 3-column piers (columns 1.6 m in height from pile cap to crosshead), namely, the Melbourne and Tooradin piers. Piers and abutment are founded on precast PSC piles. The upper portions of these piles extend above the low water level, and are concealed by precast concrete skirting panels, which are attached onto the pilecap.

Due to the deterioration of the reinforced concrete elements, major remedial works were undertaken in three stages in 1991 and 1992, which included replacement of deteriorated deck U-slabs with new U-slabs and prestressed planks the vicinity of the downstream footpath, patch repair of concrete columns and crossheads construction of a deck overlay and application of protective coatings (Ref.1).

The patch repairs included removal of concrete cover, cleaning and priming of the reinforcing bars and application of substrate bonding coat before reinstating the cover concrete with a polymer modified cementitious repair mortar.

Two generic types of protective coating were applied to the substructure elements around July 1992 to enable a long-term assessment of their effectiveness in limiting future chloride induced corrosion. A polymer modified cementitious coating was applied to the Melbourne pier, and a three-part epoxy coating was applied to the Tooradin pier. An area of 0.4 m $\times$ 0.9 m was left untreated, as control areas, immediately below the crosshead on each downstream column for subsequent comparison. A dual silane impregnation plus acrylic anti-carbonation coating system was applied to the superstructure elements.

During the remedial work, Ag/AgCl electrodes were installed for monitoring the half-cell potential of reinforcement, and stainless steel pins were installed in concrete for measuring concrete resistivity. The internal Ag/AgCl electrodes (corresponding to the points shown in Ref.1) were connected by cords leading to monitoring boxes placed at the upper edge of the crossheads (Refer Fig. 1 and Fig. 7). The locations of the resistivity measurement are also numbered accordingly. Monitoring of potentials and resistivities commenced in July 1991.

The bridge was again assessed as part of a recent in investigation in November 2002 (Ref.2). The upper part of the columns was considered as atmospheric zone and the lower part as tidal zone. Sampling and measurement at “tidal zone” were taken at 0.3 m above the pile-cap top surface.

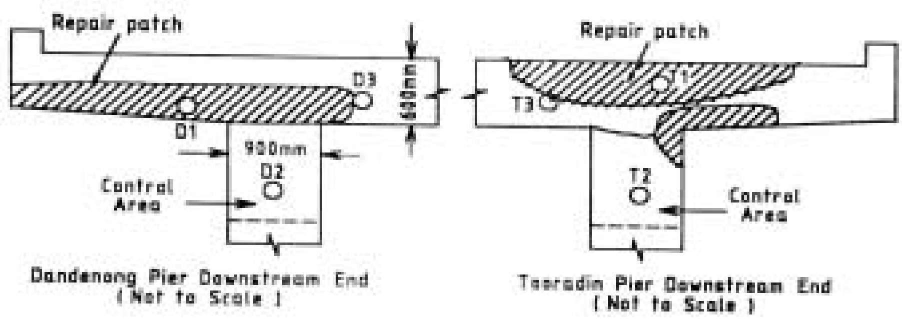

Figure 1: Location of Ag/AgCl Reference Electrodes and Patch Repairs

2. DIAGNOSTIC ASSESSMENT

Visual Inspection: Surface defects, including cracks, rust stain, spalling, and any other type of defect encountered have been summarised and illustrated for representative areas. Photographs were taken to record deteriorations, as well as the general features of the whole structure.

Delamination Test: A delamination survey was carried out by sounding technique on all the assessed areas.

Resistivity Measurement: A resistivity metre MEGGER DET 5/4R was used to determine the resistance on the installed four steel pins (spacing 5 cm).

Half-Cell Potential Testing via Existing Connections: A high internal impedance multimeter, LG DM-341, was used to measure the potential difference between the two terminals of each box.

Half-Cell Potential Mapping: A computerised half-cell potential testing instrument GD3000 was used. Half-cell potential survey was undertaken on 20 × 20 cm grids. The half-cell potential survey was made in accordance with ASTM C876, and the final output presented in the form of a potential map of the elements concerned.

Corrosion Rate of Reinforcing Steel: An electrochemical assessment of corrosion current density of reinforcement was made at selected areas where the half-cell potential evaluation indicate a high probability of corrosion activity, and at randomly selected areas representative of sound concrete. The implication of the corrosion current density can be categorised as in Table 1.

Table 1: Corrosion Current Densities for Corrosion Rate Assessment

Corrosion current density, i_{corr} (µA/cm^2)	**Corrosion rate category**
< 0.1	no corrosion expected
0.1 to 0.5	low to moderate rate
0.5 to 1.0	moderate to high rate
> 1.0	high rate

Theoretically, for steel, a corrosion current density of 1.0 µA/cm^2 is equivalent to a corrosion rate of 11.6 µm/year.

In this investigation, the corrosion current density (rate of corrosion) was determined by using a Cu-$CuSO_4$ electrode equipped with a sensor controlled guard ring to confine the area of steel bar under test.

Concrete Powder Sampling, Analysis and Penetration Rates: Concrete powders for chloride content analysis were taken by using an impact drill, from the concrete into a depth of about 80 mm at increments of about 20 mm. All drilled holes were repaired by a rapid setting marine cement mortar.

To estimate the diffusion coefficient of Cl^- in concrete and times to initiation of corrosion, the data are fitted to the theoretical equation, known as Fick's Second Law of Diffusion:

$$C = C_i + (C_S - C_i)erfc(x/\sqrt{4Dt})$$

where C is the Cl^- content at depth x, after the concrete has been exposed to a constant Cl^- environment for a period of t. C_S represents the content of chloride ions at concrete surface, and C_i that initially present in the concrete. D is the apparent diffusion coefficient.

3. RESULTS OF INVESTIGATION

3.1 Visual Inspection

Some minor rust stains and minor cracks were observed on columns. Some fine cracks were noted on the surface of the legs of the deck U-slabs, which had also previously been reported (Ref.1). It is hard to tell whether they have developed further. Nevertheless, there are no rust stains or lime leaching associated with the cracks, at the present time.

Some delamination has occurred both on non-repaired (in particular uncoated control areas) and some repaired areas (especially adjacent to uncoated control areas), which is almost the same as that reported in 2001 (Ref.3), although some of the areas are now slightly larger.

The epoxy coating on Column 3 of the Tooradin Pier (Pier 2) has bulged and separated from the base concrete, which was also reported previously (Ref.3). The epoxy layer on the pilecap has disintegrated (Figure 7). This system, which does not allow moisture to escape, appears to be unsuitable for this bridge, because the columns are constantly immersed in water. Pressure build up behind the epoxy layer finally leads to its separation from the concrete surface.

Resistivity measurements pins at the lower rows have severely corroded, and some of the pins have totally disintegrated (Figure 8). It would be difficult to perform the test on those pins in the future.

The copper wire connecting the half-cell potential box on Column 1 of the Melbourne Pier (Pier 1) (for Point 4 and 5) has broken, the tip of the wire is barely in touch with the reference pin.

3.2 Resistivity of Concrete

The resistivity results have been quite consistent in the past years, as shown in Figures 2-4. A slight increase in the resistivity is shown in all points excepting the Tooradin Pier Column 3.

Figures 1 and 2 clearly show that the resistivities of the upper parts of the columns for both piers range between 20,000 ohm cm to 40,000 ohm cm, which would indicate that the vulnerability of these areas of the columns to corrosion or the rate of corrosion supported would be low to very low. Resistivity measurements at “tidal zone”, i.e. at 0.3 m above the pile-cap top surface are between 10,000 ohm cm to 20,000 ohm cm which are likely to support low to moderate corrosion rates. The lower resistivity measurements are consistent with more negative electropotential measurements and the low to moderate corrosion current densities recorded.

All resistivity measurements at the Tooradin Pier crosshead (Fig 3) both within and outside the patch repairs, range between 20,000 ohm cm and 90,000 ohm cm, which would indicate low vulnerability to corrosion. The lower resistivity pins however, namely, pins 3-8 within the repair and pins 3-6 and 3-9 below the repair range between 10,000 ohm cm and 20,000 ohm cm which would most likely reflect the influence imparted by the delamination detected in the unprotected control area immediately below the location of these resistivity pins.

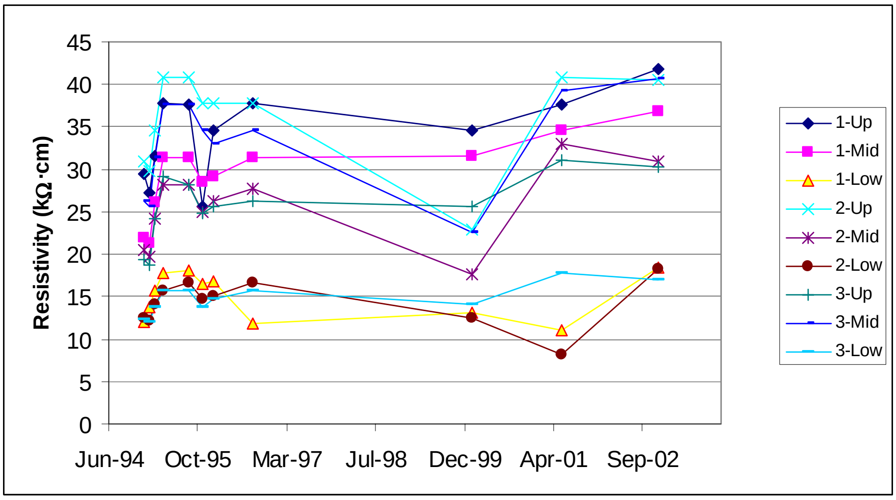

Figure 2 : Resistivity of concrete for Melbourne Pier columns

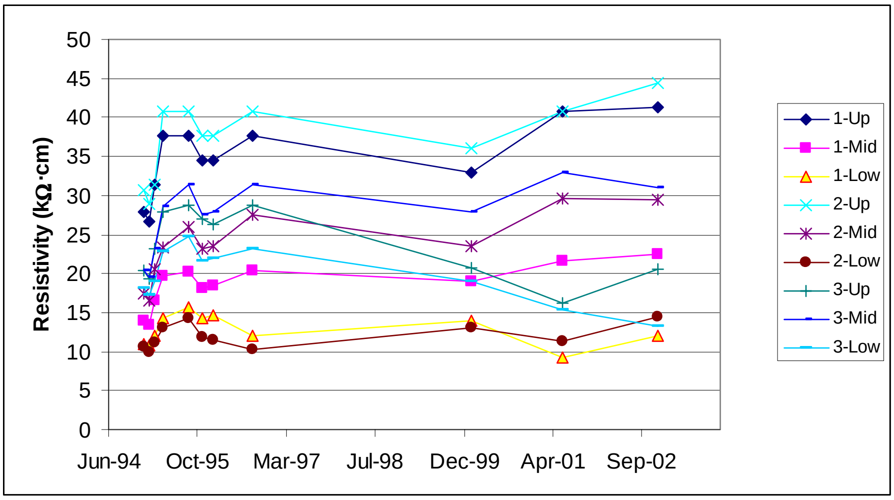

Figure 3 : Resistivity of concrete for Tooradin Pier columns

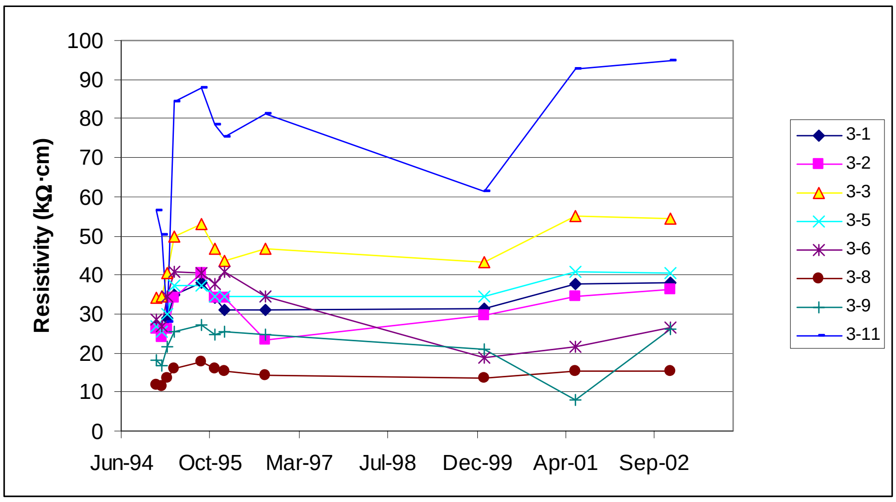

Figure 4 : Resistivity of concrete for Tooradin Pier crosshead

3.3 Half-Cell Potential of Installed Electrodes

The readings for the Melbourne Pier are all more positive than the 2001 results. The measurement on the Tooradin Pier showed more negative potential readings than those reported in 2001 (Ref.3) in two of the three locations, as shown in the following table.

Table 2: Internal Ag/AgCl sensors potential (mV)

This Investigation			Values estimated from 2001 investigation (Ref.3)	
Location	**Area**			
Melbourne Pier:D1	Repair	1	D1	-310
Melbourne Pier:D2	Control	- 60	D2	-159
Melbourne Pier:D3	Interface	- 26	D3	-284
Melbourne Pier:D4	Position Unknown	- 288	D4	-310
Melbourne Pier:D5	Position Unknown	- 309	D5	-315
Tooradin Pier : T1	Repair	- 177	T1	-241
Tooradin Pier : T2	Control	- 202	T2	-151
Tooradin Pier : T3	Interface	- 125	T3	-36

The values obtained in this investigation (Ref.2) and those that have been estimated from the 2001 investigation (Ref. 3) are summarised in Figure 5. It can be seen that the half-cell potential determined in June 2001 was substantially different compared to those determined before and after. It is thus concluded that the potentia measurement results of 2001 may not be stable due to seasonal variations, or faulty connections.

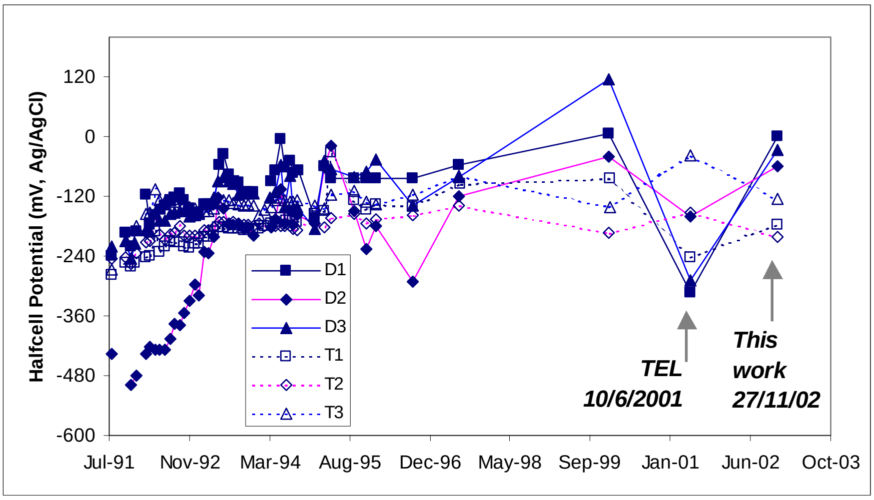

Figure 5 : Half-cell potential of the internal sensors. "D" = Melbourne Pier (Pier 1); "T"= Tooradin Pier (Pier 2)

Considering that some of the results of January 2001 may be unstable, one trend, which can be drawn, is that the steel in the unprotected control (reference) areas is the most active compared with repaired areas and the areas adjacent to the repair, for both the Melbourne Pier and Tooradin Pier. This trend has been consistent since September 1996 (excepting June 2001). The absolute values, in terms of steel corrosion activity, indicate that steel in the monitored area in the Melbourne Pier is unlikely to be corroding, whereas the corrosion activity of that in the Tooradin Pier is uncertain–to–possible in the lower part due to the effects of the large delamination within the unprotected control area immediately below the large repair area on the Tooradin Pier crosshead.

At the Dandenong Pier the half-cell potential in the repaired area (D1) is less negative than that in the area adjacent to the repair. The relative difference in the half-cell potential between D1 and the reference (D2) has been quite stable since January 1997.

Half-cell potential measurements of 1996, 1997 and 2002 (Ref.2) for D3 (the area adjacent to the repair) were more negative than D1, and the potential difference between D1 and D3 was very small. Measurement in 2000 showed that D3 was much more positive (being +114 mV, compared to -81 mV in June'97 and -288 mV in June'01). Considering the consistent results of 1996, 1997 and 2002, one can the draw conclusion that the half-cell potential at D2 is more negative than that at D1, which indicates that the repair promoted the corrosion of steel in the adjacent unrepaired/unprotected control concrete; the extent being very small.

For the Tooradin Pier the half-cell potential T1 is more negative than that at T3, which is the opposite of the case at the Melbourne Pier. It may be assumed that the steel in the repaired area may be somewhat active (in terms of corrosion) than that in the adjacent area. However, such a trend is not certain when one looks at the records of past 6 years. Nevertheless, the difference in the half-cell potential between T1 and T3 is small.

Regarding the question of whether or not the repair affects the corrosion of reinforcing steel, it is worthwhile to note that the half-cell potential values of the monitored areas indicate that corrosion is unlikely for the Melbourne Pier and that corrosion is uncertain/possible for the Tooradin Pier. The relative potential difference between the point in the repaired area and that in the adjacent area is too small to be considered significant in both piers. The corrosion brought about by the change of environment (repair), which may be attributed to the macro-cell corrosion, is not significant. This is also shown by the low corrosion current density measured in this inspection.

3.4 Half-Cell Potential Mapping and Corrosion Rate of Reinforcing Steel

Half-cell potential was determined on 20 cm $\times$ 20 cm grids by an Ag-AgCl electrode. The results are converted to CSE for reporting. The details of potential mapping and corrosion current density are shown in Table 3 and Table 4. The half-cell potential of Column 3 of the Melbourne Pier in areas under the waterline is more negative than -350 mV CSE, which indicates that the reinforcing steel bars are likely to be corroding, although this is not fully supported by either the resistivities (except lower portion) or the measured corrosion rates (0.06 to 0.1 $\mu A/cm^2$) shown in Table 6. The condition of Column 3 of the Tooradin Pier appears to be worse, indicating that the reinforcement bars of the whole column are likely to be corroding, although this is not fully supported by the measured resistivities with the exception of the lower portion of the column. The corrosion current density measured on the column surface is around 0.1 to 0.3 $\mu A/cm^2$, which indicates a slow to moderate corrosion rate. It is worth noting that the uncoated control areas of 0.4 m $\times$ 0.9 m, would have some adverse effect on the coated areas above and below, as it is likely that moisture would permeate through to the coated areas thus affecting the moisture condition of the concrete behind the coatings and any associated electrochemical processes.

The reinforcement bars in crossheads, have half-cell potential values more positive than -350 mV CSE, and their corrosion activity is uncertain. Comparatively, the part directly above the downstream columns is more likely to be corrosion active. The corrosion current density is still very low, being about 0.02 μA/cm^2 (corresponding direct potential of –153 mV CSE), which indicates that steel corrosion is insignificant under the measured area.

Table 3: Half-Cell Potential and Corrosion Rate Melbourne Pier Col. 3 and Crosshead 1 Tooradin Face Polymer Modified Cementitious Coating

Melbourne Pier Crosshead 1, C3	**CSE (mV)**	**X1**	**X2**	**X3**	**X4**	**X5**	**X6**
Tooradin Face	Y1	-153	-222	-261	-214	-65	-22
27/11/02	Y2	-176	-182	-250	-151	-68	-76
X-distance 30 cm	Y3	-110	-132	-300	-201	-193	-160
Y-distance 30 cm	i_{corr} = 0.02 μA/cm^2						i_{corr} = 0.02 μA/cm^2

Melbourne Pier (P1)C3	**CSE (mV)**	**X1**	**X2**	**X3**	**X4**
Tooradin Face	Y1	-417	-338	-263	-318
27/11/02 (i_{corr} = 0.11 μA/cm^2) control area	Y2	-348	-218	-265	-438
	Y3	-259	-242	-276	-335
X-distance 20 cm	Y4	-267	-255	-240	-262
Y-distance 20 cm	Y5	-367	-367	-380	-359
i_{corr} = 0.32 μA/cm^2	Y6	-497	-498	-510	-502

Table 4:Half-Cell Potential and Corrosion Rate Tooradin Pier Col. 3 and Crosshead 1 Melbourne Face Epoxy coated

Tooradin Pier Crosshead 2	**CSE (mV)**	**X1**	**X2**	**X3**	**X4**	**X5**	**X6**
Melbourne Face	Y1	-96	-66	-67	-79	-72	-77
27/11/02	Y2	-104	-68	-69	-81	-80	-119
X-distance 20 cm	Y3	-110	-90	-96	-135	-140	-164
Y-distance 20 cm		X7	X8	X9	X10	X11	X12
	Y1	-89	-156	-153	-149	-99	-99
	Y2	-116	-161	-236	-311	-281	-250
	Y3	-204	-202	-179	-262	-273	-281

Tooradin Pier (P2) C3	**CSE (mV)**	**X1**	**X2**	**X3**	**X4**
Melbourne Face	Y1	-220	-347	-337	-253
27/11/02	Y2	-379	-466	-404	-424
(i_{corr} = 0.10 μA/cm^2) uncoated control area	Y3	-323	-379	-449	-538
X-distance 20 cm	Y4	-346	-379	-369	-500
Y-distance 20 cm	Y5	-436	-339	-348	-353
	Y6	-422	-381	-349	-350
	Y7	-448	-414	-355	-389
	Y8	-464	-434	-369	-413
(i_{corr}=0.06 μA/cm^2) 0.3 m above pilecap	Y9	-460	-461	-403	-445

At the locations of internal electrode indicated by the 2001 monitoring (Ref.3), the results obtained by the external measurements are summarised in Table 5.

The GD3000 (computerised half cell potential measuring instrument) results are comparable to the values measured on the internal sensor terminals. These confirm that the results of June 2001 may not be fully stable. The GD3000 result for the cell DP2 is more negative than that of the internal sensor, and the GECOR result (measured nearer to the column edge) is even more negative. The corrosion activity in this area possibly varies considerably from position to position as indicated by the half-cell potential contour. Seasonal variation of the concrete environment may have contributed to the differences in potential measured at different times.

Table 5: External Potential Measurements at Internal Electrode Locations(mV, Ag/AgCl)

Element and Location	**June 2001 (Ref.3)**	**This Investigation (Nov' 02, Ref.2)**		
		Internal Sensor	**GD3000 ***	**GECOR***
Melbourne x-head-repaired zone(D1)	-310 mV	1	-28	-31
Melbourne x-head- beside repair(D3)	-284 mV	-26	-2	67
Melbourne Pier Col.3-control(D2)	-159 mV	-60	-137	-243
Tooradin x-head -repaired zone(T1)	-241 mV	-177	-174	
Tooradin x-head –beside repair (T3)	-151 mV	-125	-82	
Tooradin Pier Col.3-control(T2)	-36 mV	-202	-275	-211

* GD3000 = the average results of the area measured by GD3000; GECOR = potential measured by the corrosion current density measurement device at the mark nearest to the 2001 locations.

Table 6: Corrosion Current Density Results

ID	**Location**	***i_{corr}* ($\mu A/cm^2$)**	**CSE (mV)**
Melbourne Pier: Icoor 1	Column 3 Tooradin face, 1.4 m above pilecap	0.112	-365
Melbourne Pier: Icoor 2	Crosshead Tooradin face, near end	0.019	-153
Melbourne Pier: Icoor 3	Crosshead Tooradin face, over inner side of Column 3	0.019	-153
Melbourne Pier: Icoor 4	Column 3 inland face, 0.3 m above pilecap	0.323	-352
Tooradin Pier: Icoor 1	Column 3 Melbourne face, 1.2 m above pilecap	0.098	-333
Tooradin Pier: Icoor 2	Column 3 Tooradin face, 0.3 m above pilecap	0.055	-411

3.5 Chloride Profiles

The acid extractable chloride content was determined for each sample, and cement content determined for three samples representative of Column 3 of the Melbourne Pier (Pier 1), Column 3 of the Tooradin Pier and deck plank, respectively. Sample *P2C3 low* (Tooradin Pier (Pier 2) Column 3 at 0.3 m above pilecap) was sampled in the surface-coated area, whereas the others are not-coated areas. The results are shown in Table 8. Chloride test results from the 2001 investigation (Ref. 3) are also presented.

The chloride content at the reinforcement depth (see Table 7 below for the cover thickness) is 0.7 – 1.2 % by cement mass which has well exceeded the threshold value (0.4%) for initiation of steel corrosion in concrete. However, it should be noted that these chloride levels commenced from a previously high base prior to the application of the protective coatings. This indicates significant chloride ingress into the column. The reinforcing steel is, therefore, expected to have been affected by the chloride.

Table 7: Relevant Cover Thicknesses

Face	To main bar (mm)	To stirrup (mm)
Column-914 mm face	88.9	82.55
Column-406 mm face	76.2	69.85

Note: These are the cover thicknesses according to the design drawings

It is important to emphasise that the chloride levels at the reinforcement depth based on the 2001 test results for the unprotected control area of the Melbourne Pier (downstream column 3) was recorded at 2.5% by weight of cement compared to 0.84% - 1.21% by weight of cement recorded at 1.3m and 0.3m respectively above the pilecap on the coated (polymer modified cementitious coating) parts of column 3 of the Melbourne Pier. Te corresponding chloride levels at the reinforcement depth for the unprotected control area of the Tooradin Pier (downstream column 3) was recorded at 3.6 % by weight of cement, compared to 0.72%-0.98% by weight of cement recorded at 1.3m and 0.3 m respectively above the pilecap on the coated (three part epoxy coating) parts of column 3 of the Tooradin Pier.

This clearly shows that both the polymer modified cementitious coating and the epoxy coating have been relatively effective in limiting the ingress of chlorides into the concrete piers, despite their shortcomings which may be attributed to mixing, application and curing practices.

The chloride levels recorded in the crosshead patch repair of the Melbourne Pier (above column 3) in 2001 (Ref.3) given in Table 8 range from 0.6% at he surface to 0.07 % by weight of cement at the steel reinforcement. It is evident that the composite of the repair material and the polymer modified cementitious coating system used demonstrates superior diffusion properties to the original parent concrete. As previously noted, there was approximately one year's delay between the concrete repairs and the application of the coating, during which time at least some of the observed chloride ingress would have occurred. This further indicates in turn that the applied coating is acting as a reasonably effective additional barrier at the sample location.

The proprietary two-pack polymer modified cementitious coating has often been found to be an effective chloride barrier. However, previous experience with this system has shown that its resistance to chloride penetration is dependent on the effectiveness of curing provided at early age.

Analysis of the chloride profiles gives the apparent diffusion coefficient $D = 0.5 - 1.0 \times 10^{-12}$ m^2/s, which is similar to that estimated by the 2001 investigation.

Table 8: Chloride Content at Various Locations from 2001 and 2002 Investigations

ID	**Location**	**From (mm)**	**To (mm)**	**Cl^-/sample, %**	**Cl^-/cement, %**
P1C3*l*	Melbourne Pier(Pier1): Column 3 Melbourne face 0.30 m above pilecap	0	12	1.40	10.69
		12	40	0.38	2.90
		40	60	0.25	1.91
		60	82	0.16	1.21
P1C3*h*	Melbourne Pier(Pier1): Column 3 Melbourne face 1.30 m above pilecap	0	17	0.29	2.21
		17	29	0.24	1.83
		29	53	0.14	1.07
		53	85	0.11	0.84
P2C3*l*	Tooradin Pier (Pier 2): Column 3 Tooradin face 0.30 m above pilecap	0	10	0.2	1.15
		10	32	0.071	0.41
		32	59	0.19	1.09
		59	85	0.17	0.98
P2C3*h*	Tooradin Pier (Pier 2): Column 3 Inland face 1.30 m above pilecap	0	15	0.33	1.90
		15	32	0.27	1.55
		32	52	0.17	0.98
		52	79	0.13	0.72
S1P-1	Span 1 Plank (downstream) sea face 1.4 m from Melbourne abutment	0	18	0.016	0.08
		18	32	0.0095	0.05
		32	49	0.018	0.09
		49	75	0.0065	0.03
S1P-2	the same as above, 3.1 m from the abutment	0	10	0.019	0.10
		10	38	0.0055	0.03
		38	61	0.006	0.03
P21428 (2001)	Melbourne Pier, Column 3 x-head, inside repair	0	10		0.63
		10	30		0.56
		30	50		0.07
P21429 (2001)	Melbourne Pier, Column 3 **unprotected control area**	0	10		4.27
		10	30		2.17
		30	50		2.41
P21430 (2001)	Tooradin Pier, Column 3, x-head, lower part of repair	0	10		1.33
		10	30		1.54
		30	50		0.77
P21431 (2001)	Tooradin Pier, Column 3, **unprotected control area**	0	10		3.43
		10	30		2.73
		30	50		3.36

The predicted ages at initiation of corrosion indicate that all of the reinforcement contained in the unprotected (uncoated) original control concrete at less than 90 mm cover should now be actively corroding. This of course is consistent with the observed delaminations in the control areas. Active corrosion is not expected in the Melbourne Pier repaired areas until approximately 40 to 50 years from now. This expectation is somewhat inconsistent with the occurrence of a small delamination at the bottom part of the repaired area on the crosshead above column 3, adjoining the delaminated unprotected control area. Considering the overall good performance of this repair, this small delamination is most likely to be associated with the disruption associated with the delamination of the adjacent parent concrete within the control area. It is indeed considered that the unprotected control areas below the large crosshead repairs are likely to suffering incipient anode effects, as the repaired area is acting as a large cathode with the unprotected area becoming anodic and thus more prone to corrosion. It is worth noting that no such incipient anode effects have been observed for monitored zones beside the repairs, which were coated with protective coatings.

In contrast to the Melbourne Pier, chloride levels recorded in 2001 at the bottom part of the crosshead repair above the control area of column 3, Tooradin Pier, range between 1.33 % at the concrete surface to 0.77 % by weight of cement at the depth of the steel reinforcement. This is consistent with the resistivity of the repair area in this vicinity of about 15,000 ohm cm as recorded at resistivity pin 3-8. On the other hand the half-cell potentials in this general vicinity are in the order of -110 mV to -300 mV and the corrosion current density in the order of 0.02 $\mu A/cm^2$. It is very likely that the delamination in the control area below is also causing disruption in the lower zones of the concrete repair. It should also be noted that the internal half -cell monitoring probe within the patch repair (TP1) has recorded a potential of -177 mV indicating reduced probability of active corrosion. The significant passive potential of -125 mV observed for half-cell TP3, embedded in parent concrete adjacent to the repair further confirms that the repairs do not adversely affect the adjacent coated concrete in contrast to the unprotected control areas. Local access of water through defects in the epoxy coating film may be affecting the potential corrosion activity and the distribution of anodic and cathodic areas.

The chloride content in the replacement prestressed deck planks (downstream surface) is very low, being 0.08% to 0.10% by weight of cement at the surface. These deck planks were protected with a dual silane and anti-carbonation coating system since July 1992. Comparison of the columns and the deck components show that as a result of chloride ingress into the columns, the half-cell potential values are generally much more negative, and the corrosion rates much higher than those for the deck planks. Differences between the two columns are not very pronounced.

Based on the relatively low chloride levels the steel reinforcement in the deck planks is likely to remain passive for at least 80 to 100 years, provided the coating system is maintained. It is not possible to predict the time because the maximum chloride content in the plank is still below the threshold level. Future monitoring would be necessary for reliable prediction. However, given that a significant proportion of the measured chloride may have entered the planks in the interval between installation and application of the coating system, the data demonstrates that the applied coating system is effectively restricting chloride ingress into the deck soffit.

4. DISCUSSION

4.1 Structural Elements and Repair

Corrosion of reinforcing steel has caused some concrete cracking in the columns of Sawtell Inlet Bridge in both the original concrete and in the vicinity of some repaired areas. The surface of the unprotected control areas in the downstream columns (column 3) and a small number of areas in the crossheads has shown some concrete delamination, the extent of which is the same as last reported in 2001 (Ref.3). Some of the cracks and delamination are actually observed in the cementitious coating of the Melbourne Pier or the epoxy coating of the Tooradin Pier, and they may be limited to the coating material, or between the coating material and the concrete substrate.

The chloride content at the reinforcement level for all columns has well exceeded the threshold level for initiation of steel corrosion. More deterioration can be anticipated, based on the chloride content of the concrete. However, at present, the corrosion rate can be categorised as “low-to-moderate”.

Fine cracks were observed on the soffit of the old deck U-slabs, but the extent of the cracking appears to be insignificant. The relatively new deck planks assessed in this investigation (the downstream planks) have not shown any deterioration, and chloride content in the concrete was very low. It is expected that it will continue to perform well in the future. Generally, the high chloride content in the columns was associated with more negative half-cell potentials and larger corrosion rates of the steel reinforcement bars, compared to the situation with the deck planks which had very low chloride contents and in which the steel is not corroding.

The epoxy coating on the downstream column of the Tooradin pier has deteriorated and has peeled off from the pilecap, and has bulged at the lower part of the column. It would appear that vapour pressure build up behind the coating could have caused it to separate from the concrete substrate. Some peeling of the cementitious coating has also been noted in some areas, particularly at the lower sections of columns and the top of the pilecap.

Although both coatings are approaching the end of their serviceable life, particularly the three-part epoxy coating, they appear to be capable of providing adequate protection to the chloride exposed concrete surfaces. However, it is considered that the surface preparation and surface condition, mixing of the materials, application and curing of these coatings together with other operational practices (i.e. temperatures, dew points, relative humidities etc) may have a major impact on the ongoing performance and their ability to achieve their intended service life. The two-pack polymer modified cementitious coating material was found to be easier to handle and mix and therefore much easier to roller or brush apply to perhaps a uniform quality and thickness (not measured at the time), although its curing which required the application of a curing compound may have been somewhat problematic. On the other hand, the three part epoxy coating was very difficult to handle and mix and also difficult to apply at a uniform quality and thickness as it was hand applied and had a tendency to slump away from the concrete surfaces. As such the application of this system should be modified, otherwise its application to concrete elements in tidal zone may have to be reconsidered.

4.2 Monitoring Systems

The embedded Ag/AgCl potential sensors in the Melbourne Pier show somewhat unstable readings in the current survey, the reasons for which is not clear. The sensors in Tooradin

Pier appear to be functioning properly. Summarising the results of the past few years, it can be concluded that the steel in the control area (both piers) has a more negative half-cell potential than that in the repaired area and in the area adjacent to the repair. This indicates that the steel in the repaired and the adjacent coated areas are in a safer condition than that in the unprotected control areas. In general, the half-cell potential change with time appears to be insignificant (excepting the measurement in June 2001, which is considered to be inconsistent with other measurements), and at this stage it is unlikely that the repairs have caused any significant corrosion.

The externally measured half-cell potential is comparable to that of the internal sensors at almost all the locations. But a larger difference has been observed for the steel around the sensor in the unprotected control area of the Melbourne Pier. It is probable that the corrosion state of the reinforcement in this area is more complicated, which is reflected in the half-cell potential mapping conducted in this inspection. In general, the above results indicate that the sensors are working properly.

The resistivity arrays showed similar readings as those assessed in 2001 and earlier. The resistivity tested at most of the locations has been increasing, though only slightly, in the past few years. However, the resistivity of the lower portion of the downstream column 3 of the Tooradin pier has shown a decrease with time since 1999 (there was no data between 1996 and 1999). The lower shift of the resistivity could have been caused by the increased penetration of seawater into the concrete, which is consistent with the damage of the of the epoxy coating applied on this column.

Finally, there are a number of resistivity measurement pins in the crossheads, which show extremely large resistance. This may not be related to the concrete properties (although they were also observed in the patch repairs from the start), but rather to the condition of the pins. The resistivity pins at the lower positions in the columns (about 0.3 m high above pilecap top surface) have severely corroded, and future measurement at these points will be difficult. These pins should be checked and fixed in order to make reliable measurements in the future.

5. SUMMARY AND CONCLUSIONS

Recent investigations and assessments reveal that although after nearly 12 years of service a proportion of the concrete repairs and protective coatings are approaching the need for re-intervention in this very aggressive marine environment, overall the combination of some repairs and associated coatings has proven to have performed in a reasonably satisfactory manner. Analysis of selected elements has shown that sufficient amounts of chloride ions have penetrated to the level of steel reinforcement in the columns, although these commenced from an already high base prior to the application of the protective coatings. This has been associated with low to moderate corrosion rate of the steel, as indicated by the half-cell potential values and the direct measurement of corrosion rate of the steel, particularly in the unprotected control areas and the lower portions of the columns. Prestressed concrete deck planks that had been installed as part of the rehabilitation process showed that the extent of chloride ingress and corrosion was insignificant. This is due to the dual protective coating (silane and anti-carbonation coating) applied to the planks after installation in the bridge.

The epoxy coating of some of the elements appears to have peeled off the concrete substrate, and is now ineffective with respect to prevention of chloride ingress in some areas tested, whereas the cementitious coating has also peeled off in some areas, particularly at the lower sections of columns and the top of the pilecap. However, it may be concluded that both the polymer modified cementitious coating and the epoxy coating have been relatively effective in limiting the ingress of chlorides into the concrete piers, despite their shortcomings which may be attributed to mixing, application, curing and other operational practices. The application of polymer modified cementitious repair material does not seem to have caused or enhanced macrocell corrosion in the neighbouring coated areas. Because of significant chloride levels in the columns, consideration may have to be given to the installation of a CP system (vs. ongoing patch repairs and coatings) to protect the piers from further progress of corrosion, based on a life cycle costing analysis.

REFERENCES

1. ANDREWS-PHAEDONOS, F. (1994), "Rehabilitation and subsequent monitoring of Sawtells Inlet Bridge, South Gippsland Highway, Tooradin, Victoria," *Proceedings 17th ARRB Conference*, Part 4, pp.11-27.

2. SHAYAN, A. and XU, A. (2003), :Assessment of Sawtells Inlet Bridge", ARRB Contract Report No. RC2727, for VicRoads.

3. TAYWOOD ENGINEERING LIMITED (2001), *Sawtell's Inlet Bridge Corrosion Monitoring, June 2001 Update*. Report No. 1303/01/8857.

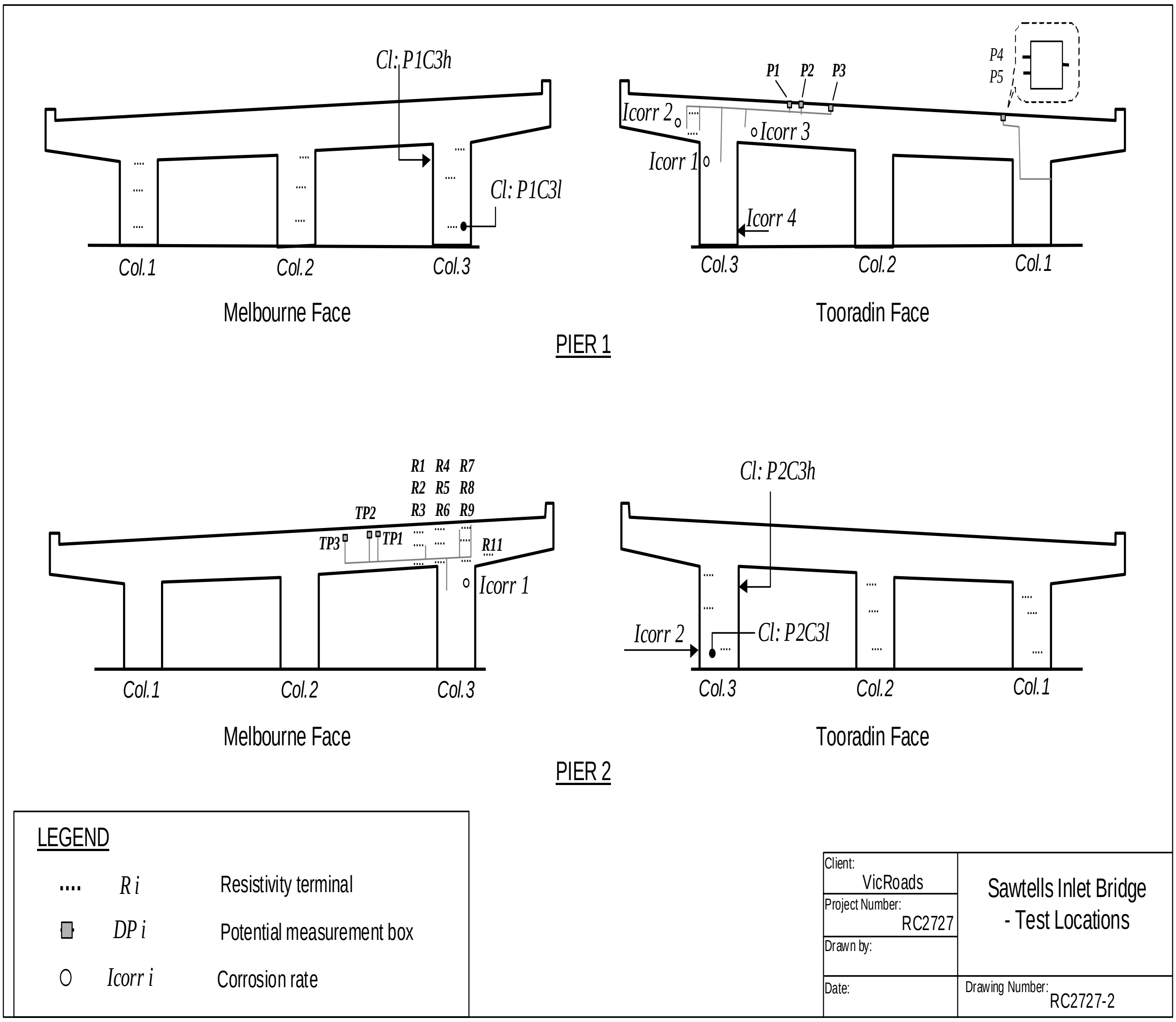

Figure 6 : Sawtells Inlet Bridge Test Locations, November 2002

Figure7:The investigated piers(left:Melbourne Pier;right:Tooradin Pier)at low tide,Nov' 02

Figure 8 : Columns 1 and 2 of Melbourne Pier (Pier 1), Melbourne face

Figure 9 : Columns 2 and 3 of Tooradin Pier (Pier 2), Melbourne face