

# Condition assessment of a reinforced concrete jetty structure, its load capacity and suggested rehabilitation strategy

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## ABSTRACT

A reinforced concrete jetty structure built in 1970 has developed severe deterioration due to chloride-induced corrosion of the steel reinforcement bars in the piles and cross-heads. The piles have shown severe vertical cracking and the crossheads severe horizontal cracking and delamination of the cover concrete at the soffit. After a thorough visual inspection, concrete cores were extracted from representative elements and tested for strength, density, and chloride penetration. Electrochemical properties of steel, including half-cell potential and corrosion rate were determined, and concrete cover thickness measured on site to map the corrosion status of the various elements. The load capacity of the jetty was analysed based on the original design and taking into account the observed deterioration of the concrete and the corrosion of the steel reinforcing bars, and the trend of decay in the load capacity determined. In spite of severe deterioration of crossheads, the load bearing capacity remains adequate for the current load rating of 7 tonnes and 10 tonnes for single axle and double axle, respectively. A rehabilitation strategy was suggested based on the overall results, comprising patch repair to deteriorated concrete elements followed by cathodic protection of steel bars.

## 1. INTRODUCTION

The jetty structure (Figure 1) was constructed in 1970, and consists of precast concrete piles cast-in-situ crossheads, precast prestressed concrete double-T deck planks with cast-in-situ concrete overlay. It is about 445 m long supported by 103 piers (bents) with span lengths of about 4.5 metres. There is an expansion joint every 13 to 15 bents and at each expansion joint, there are two adjoining piers standing side-by-side, such that the 103 piers comprise 97 spans. Drawings of the structure provided the following design information:

Structural element	Concrete compressive strength, 28-day, Min.	Cover to reinforcement
Double T deck plank	31 MPa (4500 psi)	38 mm
Deck overlay		50.8 mm to mesh
Crosshead	31 MPa	63.5 mm
Pile	31 MPa	57.2 mm

Documentation dated 1994 indicated that some piles and crossheads had already been repaired, and that the last load capacity assessment gave load ratings for single axle and double axle of 7 tonnes and 10 tonnes, respectively.

## **1.1 Scope of the present work**

After a visual inspection and selection of representative concrete elements, they were investigated for concrete cover thickness, delamination, concrete resistivity, half-cell potential and corrosion rate of steel reinforcement bars, chloride content profile, and compressive strength. The load capacity of the structural components was determined for the design condition as well as for the deteriorated condition to enable the estimation of the trend in the reduction of the load capacity with time.

From the general visual inspection the following areas were selected for further testing:

- Zone 1: Bent 87 where the crosshead shows reinforcement corrosion and concrete delamination;
- Zone 2: Bent 77 where the crosshead has large cracks caused by reinforcement corrosion, a common feature of the whole structure;
- Zone 3: Bent 4, repaired in 1994;
- Zone 4: Bent 68 where the crosshead of the 3-pile pier has cracks along both upper and bottom main rods caused by reinforcement corrosion;
- Zone 5: Bent 7, where the cover concrete of the crosshead soffit has spalled off and the reinforcement mesh is exposed, and where the piles have large cracks.

The piles, crosshead and part of the deck soffit including parts of the T-beams near the crosshead, were examined for all the zones mentioned above.

## **2. METHODOLOGY**

### **2.1 Visual inspection**

The inspection was conducted in a boat to locate obviously deteriorating areas where closer observations were made. Surface defects including cracks, honeycombing, rust staining, efflorescence, spalling, exposed reinforcement and any other type of defect encountered were documented and summarised, and photographically illustrated for representative areas.

To facilitate accurate assessment of load carrying capacity, the dimensions of structural elements were checked on site. The visual survey also detailed the diameter and corrosion state of any exposed reinforcement. A delamination survey was carried out on all the selected areas at the time of the inspection.

### **2.2 Covermeter survey**

After calibrating the instrument on site, covermeter measurements were made to assess the thickness of the cover concrete at all selected areas.

### **2.3 Rebound hammer test**

A Rebound hammer survey was undertaken in accordance with ASTM C805 at selected areas to assess the uniformity of concrete and to delineate regions in the structure of poor quality and or delamination. It included deteriorated areas and areas representative of sound concrete. For each area, 10 readings were taken spaced at more than 25 mm apart.

## 2.4 Half-cell potential mapping

A half-cell potential survey was undertaken in accordance with ASTM C876, to assess the corrosion activity of reinforcement on selected deteriorated areas and selected areas representative of sound concrete. The measurements were made on grid sizes, depending on the dimensions of the element concerned, so that the gradient in the potential could be clearly determined. The final output was presented in the form of a potential map of the selected element.

## 2.5 Corrosion rate of reinforcing steel and electrical resistivity of concrete

A linear polarisation device with a sensorised guard ring (Gecor 6), was used for electrochemical assessment of corrosion of reinforcement at selected areas, where the half-cell potential measurements indicated a high probability of corrosion activity. The instrument used a Cu-CuSO<sub>4</sub> electrode equipped with a sensor controlled guard ring to confine the area of steel bar under test. The resistivity of concrete was also determined at these areas by the same instrument.

The interpretation of the corrosion current density is given by Broomfield et al(1) as follows:

Corrosion current density, I <sub>corr</sub> (μA/cm <sup>2</sup> )	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<sup>2</sup> is equivalent to a corrosion rate of 11.6 μm/year.

## 2.6 Core Sampling

Concrete cores (75 mm diameter) were taken from structural members, on representative deteriorated and sound areas of concrete. According to AS 3600, three or more cores were taken from each element tested. All drill holes were repaired by a polymer modified cement mortar, including core holes and holes drilled for electrical connection to the reinforcement.

## 2.7 Laboratory testing

Laboratory testing was carried out, on the concrete core samples to determine their chloride profile, compressive strength, and density.

Chloride ion content was determined following AS 1012.20 for several depth increments in the cover concrete and beyond the steel reinforcement depth. The chloride content measured by this technique (acid digestion), gives the total chloride in the concrete. The concentration of chloride required for initiation of reinforcing steel corrosion is related to the hydroxyl ions in the pore solution, which varies depending on cement type and concrete quality. In general, a limit of 0.8 kg/m<sup>3</sup> is given by AS 3600 as the maximum acceptable Cl<sup>-</sup> content in reinforced concrete structures. The value corresponds to 0.23% Cl by mass of cement for a concrete

containing 350 kg cement/m<sup>3</sup>. Corrosion initiation of steel reinforcement has been stated to be associated with chloride contents of about 0.4% by mass of cement (BRI, UK(2) and Broomfield(3)). For a concrete containing 350 kg cement/m<sup>3</sup>, this corresponds to a chloride content of 1.4 kg/m<sup>3</sup>, as compared to the acceptable level of 0.8 kg/m<sup>3</sup>.

To estimate the diffusion coefficient of Cl<sup>-</sup> in concrete, the chloride profile data are fitted to the theoretical equation, known as Fick's Second Law of Diffusion:

$$C = C_i + (C_s - C_i) \operatorname{erfc}(x/\sqrt{4Dt}) \quad (1)$$

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

The prediction of critical Cl<sup>-</sup> migration into concrete can be made by substituting these data into the above equation and calculating the time at which the critical Cl<sup>-</sup> level is reached at the depth of the steel reinforcement in concrete. The prediction is made for both Cl<sup>-</sup> limits of AS 3600 (Cl<sup>-</sup> = 0.8 kg/m<sup>3</sup>), and Cl<sup>-</sup> = 0.4% by mass of cement.

Compressive strength was determined according to AS 1012.9 and AS 1012.14, and the wet density of concretes according to AS 1012.12.1.

## 2.8 Load carrying capacity assessment

Calculations were made on the basis of current load ratings, the structural drawings, condition of reinforcing steel, concrete delamination and concrete strength. Comparison with the acceptable live load bearing safety factor ( $\gamma_{LL} = 1.5$ ) was made to estimate the load capacity reduction, and the time when the safety factor would decrease to unacceptable levels of below 1.5.

## 3. RESULTS OF INVESTIGATION

### 3.1 Visual inspection

#### ▪ *Defects observed in piles:*

Many piles displayed large vertical cracks running parallel to the reinforcing bars at the corners of the piles (Figure 2). Rust stains are seen at some of the cracks and more at the junction of crosshead and pile. Delamination has occurred for some piles (Figure 3), where large cracks have developed at the two sides of one corner of the pile. In general, it appears that large cracks and delamination in piles mainly occur in bents near the shore. Approximate frequency of defects in the piles in the form of delamination, large cracks and rust staining, was 5%, 20% and 40%, respectively, of the total number of piles.

#### ▪ *Defects observed in crossheads:*

Reinforcement corrosion has occurred in most crossheads. Large cracking was present along the main reinforcing bars at the lower part of crossheads, about 100 mm from the soffit, and traverses the distance between the piles (Figure 4). Many crossheads have suffered delamination at this part, and large areas of spalling has occurred for some of the crossheads

(Figure 5, 6). Significant loss of steel was noted in some of the stirrups in the spalled areas, and some stirrups appeared to have broken at the soffit. A significant loss of stirrup sectional area may affect the shear capacity of the crossheads. Honeycombing was also seen at the bottom edges of some crossheads (Figure 7). The steel section loss due to corrosion, in a location where the rusted reinforcement was exposed, was about 1-1.5 mm in depth, i.e. the diameter of the main steel bars (7/8 inch bar = 22.2 mm) was reduced to about 19-20 mm, after clearing the rust.

Crossheads 1 to 5 appeared to have been repaired at the lower part, where stainless steel bolts for fixing formwork are still in place (some of the bolts on Crosshead 5 are electrically connected to the reinforcement).

Corrosion of the upper main reinforcing bars has also caused cracking, but only in some of the crossheads. Scaling and efflorescence were common on crosshead surfaces, particularly at the crack sites.

The approximate frequency of defects in the crossheads in the form of delamination, large cracks and rust stains was 15%, 90% and 90% of the total number of crossheads.

▪ ***Defects observed in deck planks:***

Some local reinforcement corrosion and small cracking have occurred at edges and corners of the deck planks (Figure 8), and on side surfaces of double T-beams. Slight scaling was evident on the top surface and soffit of the deck. Concrete coring at the top of deck exposed steel bars which showed very slight corrosion at 79 mm and 85 mm, and a light corrosion at 55 mm. At the side of a double-T plank, exposed prestressing strands (depth of 72 mm) showed no signs of corrosion.

In general, the defects observed in the structural elements, especially those in the crossheads and piles could affect the integrity of the structure. Therefore, the overall condition of the pier is rated poor.

### **3.2 Cover survey**

Concrete cover reinforcement was determined by a cover metre and by drilling to reinforcement. The cover thickness for the various elements was:

- Piles: between 70 to 90 mm.; Crossheads: between 40 to 50 mm at soffit;
- Deck soffit: between 35 to 40 mm, with cover to prestressing strands of 70 mm.

The cover thickness of crosshead is smaller than the design thickness of 2.5 inches (63.5 mm).

### **3.3 Half-cell potential of reinforcing steel**

Half-cell potential was determined on 20 cm × 20 cm or 25 cm × 25 cm grids by the Ag-AgCl electrode. The results were converted to copper-copper sulfate electrode (CSE) at 25°C as required by the ASTM standard for reporting [CSE = (Ag/AgCl – 123) mV]. The temperature at the time of test was 10°C. The results are summarised in Table 1, and an example of the half-cell potential map produced for all the elements tested is presented in Figure 9.

**Table 1: Summary of half-cell potential measurements**

Zone	Location	Element	Half-cell potential (mV)	Corrosion activity
1	Bent 87 west side	piles	-450 to -770	Very likely – high potential gradient near junction with crosshead
		crosshead	-480 to -726	Very likely as above.
2	Bent 77 west side	piles	-540 to -870	Similar to Zone 1
		crosshead	-413 to -722	Similar to Zone 1
3	Bent 4 partially repaired	piles	-214 to -530	Corrosion activity unlikely in repaired areas, but likely at the junction of repaired and un-repaired areas in both piles and crossheads
		crosshead	-110 to -470	
4	Bent 68 west side	piles a	-408 to -570	Low to moderate
		b	-314 to -491	
		crosshead	-250 to -448	Low to moderate
5	Bent 7 all areas	piles	-503 to -650	Moderate to high
		crosshead	-257 to -540	Isolated areas – low corrosion rate
6	Deck soffit	Zone 1	-360 to -592	Larger near crossheads
		Zone 2	-375 to -715	Larger near crossheads
		Zone 3	—	
		Zone 4		
		Zone 5	-70 to -298	Negligible

In Zone 1, both piles have shown half-cell potential values more negative than -450 mV CSE, and a high gradient in areas adjacent to the crosshead, indicating corrosion activity. The crosshead also showed a high corrosion activity, indicated by its very negative potential (-450 to -750 mV CSE), particularly in areas near the piles. The deck soffit showed corrosion activity near the crosshead area.

The same pattern was noted in Zone 2, where results indicated a high corrosion activity in the crosshead, and isolated corrosion spots in the deck slab and piles.

In Zone 3, the low part of the crosshead and some parts of the piles appeared to have been repaired. The half-cell potential of the repaired areas is generally more positive than -250 mV CSE, indicating that the corrosion activity has been reduced to a safe level. However, the high potential gradient between repaired and un-repaired areas suggests a high corrosion activity along the junction between repaired concrete and the original concrete.

For Zone 4, corrosion has obviously occurred where cracks appear with rust stains at the surface, but corrosion activity in the upper parts of the crosshead is not certain as the half-cell potential was more positive than -350 mV, whilst it was more negative in the lower parts.

For Zone 5, the potential map for the crosshead indicated high corrosion activity in isolated areas. The piles in Zone 5 showed a very negative half-cell potential, especially in their upper section. For the deck soffit however, corrosion may have occurred in areas near crossheads indicated by the potential gradient. However, the half-cell potential was generally more positive than -250 mV, and the possibility of active corrosion occurring at the time of measurement may not be high.

Comparing the results of this work with one conducted in 1994 by others shows that the half-cell potential has become much more negative, an indication of enhanced corrosion activity.

### 3.4 Corrosion rate of reinforcement and concrete resistivity

Table 2 summarises the measurements made on some of the elements. The concrete resistivity in the piles and crossheads was relatively low due to the ingress of chloride ions into concrete, but it was much higher for the deck which is less subject to contamination with salt water. The corrosion rate was higher where the half-cell potential and concrete resistivity were low, as expected, but this correspondence did not appear to hold for the crosshead of Bent 7. The deck at Bent 7 showed negligible corrosion activity.

*Table 2: Corrosion rate of reinforcing bars and concrete resistivity*

Zone	Location	Element	Half-cell potential* mV	Corrosion current density uA/cm <sup>2</sup>	Concrete resistivity KΩcm	Corrosion <sup>§</sup> activity
4	Bent 68 west side	crosshead	-435	0.33	6	low to moderate
5	Bent 7	piles	-585	0.60	4	moderate to high
		crosshead	-311 -420	Upper bars 0.07 Lower bars 0.05	5 6	negligible
		deck	-165	0.007	18	

\* at the side of corrosion rate measurement

§ based on the value of corrosion rate

### 3.5 Chloride Profile

The chloride profiles of the elements tested are shown in Figure 10, where the data points and the fitted curves are both shown. Table 3 presents the values of the fitted parameters  $C_s$  and  $D$ . Based on these parameters, the trends for the ingress of the critical amount of chloride into concrete have been predicted and an example for a prestressed deck beam and a crosshead are presented in Figure 11.

The  $Cl^-$  content at the concrete surface was generally high, due to the marine environment of this structure. At the depth of reinforcement,  $Cl^-$  content was higher than or near the acceptable value of  $0.8 \text{ kg/m}^3$ . The higher values refer to the deck plank soffits, and crosshead underside where the cover concrete is thin, and the chloride content is three times higher than the acceptable level.

**Table 3: Fitted values of surface chloride content and chloride diffusion coefficient**

Sample		Curve fitting analysis	
ID	Location	C <sub>s</sub> (%)	D (m <sup>2</sup> /s)
A	Deck top Bent 39-40	0.45	1.6E-13
B	Crosshead 8	0.41	1.90E-12
C	Deck beam Bent 31-32	0.67	4.76E-13
D	Crosshead 68	0.25	2.85E-12
E	Pile 87A	0.32	1.11E-12

The fitted values of the apparent diffusion coefficient ranged from  $0.2 \times 10^{-12} \text{ m}^2/\text{s}$  to  $3 \times 10^{-12} \text{ m}^2/\text{s}$ . The lower values are for the concrete of deck plank, and the higher ones for crossheads.

The predicted trends of chloride ingress (Figure 11) show that the critical chloride level for initiation of corrosion (0.4% by mass of cement) will reach the reinforcement level at around 16 years and 60 years for the crosshead and the deck respectively. Considering that the structure is 30 years old, the level in the crosshead was probably exceeded 14 years ago, and that in the deck will be reached in 30 years time.

### 3.6 Strength of concrete

The average and range of strength determined on 33 core samples were:

Deck:  $29 \pm 3 \text{ MPa}$ ; Crosshead:  $36 \pm 8 \text{ MPa}$  (all  $> 34 \text{ MPa}$ ); Pile:  $30 \pm 6 \text{ MPa}$

A rebound hammer survey was carried out in all the testing zones. The readings of the rebound hammer showed higher strength than that tested on cores, but the results largely reflected the strength variation range, when they were calibrated using areas where both tests were conducted.

## 4. LOAD CAPACITY

### 4.1 Design load capacity

The load capacity assessment was based on concrete compressive strength determined in investigation, and Reference drawings. The structure was checked for live loads of 7 tonnes and 10 tonnes imposed by single axle and double axle vehicles, respectively. The structure as designed was found to be adequate for these live loads with respect to the capacity of deck slab, deck beam, crosshead and piles.

### 4.2 Current load bearing capacity

The deterioration of concrete and corrosion of reinforcing bars indicate that the design load capacity of the various elements may have decreased. A reassessment of load capacity

showed that the load capacity was still adequate for the deck slab and deck beam, due to the small degree of deterioration in these elements.

For the crosshead, the steel bars exhibited 1.5 mm loss of steel due to corrosion and in some areas a few broken stirrups. Although the load capacity was still adequate, considering the loss of steel, the corrosion may also have caused loss of bond between the steel bars and concrete (bond appeared to be intact at the supports). This situation requires urgent attention to the repair of the element.

Calculations have shown that the dead load and live load demand (10 tonnes double axle) is 163 kN compared to the design shear capacity of 274 kN for the crosshead. It has been calculated that 45% loss in the cross-sectional area of the main reinforcing bar (or 26% loss in bar diameter) would be needed before the live load safety factor is reduced below 1.5, the minimum value demanded by the asset owner, and the crosshead becomes critical in flexure. Similarly, 65% loss in the stirrups cross-sectional area (or 40% loss in bar diameter) would be needed before the crossheads become critical in shear. The observed deterioration was generally less severe than these losses, indicating that the crossheads are satisfactory in shear and flexure under the current loading.

For the piles, it was noted that the corners of some piles were broken off due to corrosion-induced cracks on the two adjacent faces of the corners, although for most piles only one side exhibited cracking. Assuming complete loss of the cover concrete due to delamination, the capacity of the confirmed core of the pile was calculated to be 610 kN compared to the demand of 300 kN. The relatively small steel section loss in the bars does not reduce the capacity of the pile in compression to any significant extent. Therefore, the piles are considered to be satisfactory for the current load demand. Overall, the load ratings for the concrete structure remain the same as that assessed based on the inspection of May 1994, being 7 tonne and 10 tonne for Single axle and Double axle, respectively.

### **4.3 Trend of the live load capacity reduction**

It should be noted that the current load ratings of the jetty would have been made based mainly on the load capacity of the deck beams, which are critical for load assessment, but are not observed to be deteriorating. All the other structural elements had higher load capacity reserves. In our inspection, there were no obvious deterioration in the deck beams, which would imply that the load ratings should remain unchanged. It should be mentioned, nevertheless, that the safety factor for the deck beam is just above 1.5, and corrosion of reinforcement in the deck beams may lead to further reduction in load rating. Moreover, deterioration of the crossheads has made their condition more critical to the load capacity than the deck beams.

It should be pointed out that reliable prediction could only be made up to the stage when reinforcing bar section loss is less than 25%. Beyond this stage, new assessments of the condition of the piers would be necessary. Further corrosion would result in more delamination and spalling of cover concrete, and more severe loss of bond between the steel and the concrete. This is likely to cause unpredictable changes in the geometry of the structural element, which in turn can drastically change the load bearing capacity.

The moment of an element is calculated by

$$\phi M = 1.25M_{DL} + \gamma_{LL} \cdot 1.1M_{LL} \quad (2)$$

where  $\phi$  (+0.8) is the strength reduction factor,  $M_{DL}$  moment due to dead load,  $M_{LL}$  moment due to live load; factor 1.25 is due to the uncertainty in material property, 1.1 is the impact factor for live load, and  $\gamma_{LL}$  is the safety factor.  $\gamma_{LL} = 1.5$  is the minimum acceptance value for load rating. The formula for shear capacity is essentially the same, but in this case the strength reduction factor  $\phi = 0.7$ .

Based on the load distribution for single and double axle loading the  $\gamma_{LL}$  of crosshead, which is the worst deteriorated element of this structure was calculated and its values with time are plotted against time as shown in Figure 12. Alternatively, the trend can be presented in terms of axle load as shown in Figure 13 based on  $\gamma_{LL} = 1.5$ . Existing information from previous inspections were used to produce these graphs. The calculation for the present situation was based on our current results for strength of concrete and the rate of the reinforcing steel corrosion. It was also based on the current chloride profile, according to which the corrosion of reinforcing steel bars may have started in the late 1980s.

Figure 13 are load rating graphs for both the deck beams and the crossheads. Given the current condition of the crossheads and the anticipated future deterioration of these members, it is predicted that crossheads will become critical in the future while the condition of the deck beams will remain relatively stable. Hence, future life predictions are based on the condition of the crossheads as being critical. Of course, the structure should be re-inspected within 5 years to reassess the condition of its components and re-determine appropriate load ratings.

## **5. REMEDIAL OPTION**

Depending on the condition of the structure and available budget, some of the following strategies may be employed in its rehabilitation.

- No rehabilitation intervention
- Patch repair
- Cathodic protection
- Replacement with a timber structure

These options are briefly discussed below.

### **5.1 No rehabilitation strategy**

This is not a practical option for this jetty as severe deterioration has already occurred. Should no remedial action take place, the current corrosion rate indicates that the worst affected crossheads would become unsafe within 5 years, when a major rehabilitation would be needed.

### **5.2 Patch repair**

This approach is most effective before  $Cl^-$  contamination reaches the steel level. In the case of extensive contamination, successful repair can be done only if all the contaminated

concrete is removed and replaced by new concrete of appropriate composition and quality. Even the latter provides only a temporary suppression of corrosion activity, and may need replacement in the future. Repair of limited portions of contaminated concrete would fail prematurely due to corrosion induced cracking in the neighbouring area.

Patch repair is an expensive exercise. Although accurately costing of the patch repair option for this jetty is difficult an estimation based on the worst case of large cracking and delamination has been made and amounts to around \$500,000.

### **5.3 Cathodic protection (CP)**

The cost of CP will depend on the type of cathodic system applied. An appropriate system which has been used for bridge structures is the ribbon system for impressed current cathodic protection (CP). It provides a permanent protection for the normal reinforcement.

It should be mentioned that according to the present knowledge, impressed current CP is not recommended for prestressed elements and for concrete with AAR problem. Therefore, for this structure, CP is not recommended for the deck. A sacrificial anode system may be applied to the prestressed concrete structure.

For crossheads and piles, an embedded ribbon configuration is recommended, which can be applied by making 3 cuts per element such as crossheads. The cost of this system would include restoration of locally damaged concrete, but not removal of all the contaminated concrete. The budget estimate for the application of CP to 82 piles and 93 crossheads would be around \$150,000, excluding the cost for power supply and remote control. Areas of delamination of the crossheads would need to be repaired prior to the application of CP, which would increase the cost of this option. The actual cost would need to be confirmed by a contractor.

### **5.4 Replacement of the pier with a timber or concrete structure**

The concrete pier approach is 445 m long and about 4 m wide, i.e. 1780 m<sup>2</sup> surface area. Based on the unit price of \$1,500 and \$2,500 per square meter for timber and concrete, the total replacement cost would be \$2,670,000 for timber and \$4,450,000 for concrete, respectively, excluding the cost of demolition.

## **6. RECOMMENDATIONS**

It is recommended that all the delaminated and badly cracked concrete in the crossheads and piles be restored. It is further recommended to carry out cathodic protection for these crossheads and piles in the first stage to gain experience on the behaviour of the protected elements. If successful the CP could be extended to all elements with high chloride contamination. It is recommended that the deck is inspected again after 5 years. Sacrificial anode cathodic protection may be considered for the deck slabs in the future.

## 7. CONCLUSIONS

Investigation of the jetty structure has shown that the deck is in fair condition at present. However, the chloride content at the level of reinforcing bars (40 mm in average) is high and may cause reinforcement corrosion in the future. At the level of prestressing strands, the Cl<sup>-</sup> content has not yet reached the critical limit for corrosion initiation. For the prestressing strands, which are more sensitive to the pitting corrosion, the increasing Cl<sup>-</sup> content will be a major concern in the future.

A large number of crossheads are badly affected by the corrosion induced cracking and delamination, which attributed to the relatively thin concrete cover at the soffit, i.e. about 40 to 50 mm, which is low compared with the design thickness of 64 mm. The quality of concrete might also be low at the soffit of some crossheads evidenced by the occurrence of honeycombings. The patch-repaired crossheads showed a low probability of corrosion in the repaired areas, though a high corrosion activity at the border of the old and new concrete. This is the problem with patch repairing of Cl<sup>-</sup> contaminated concrete.

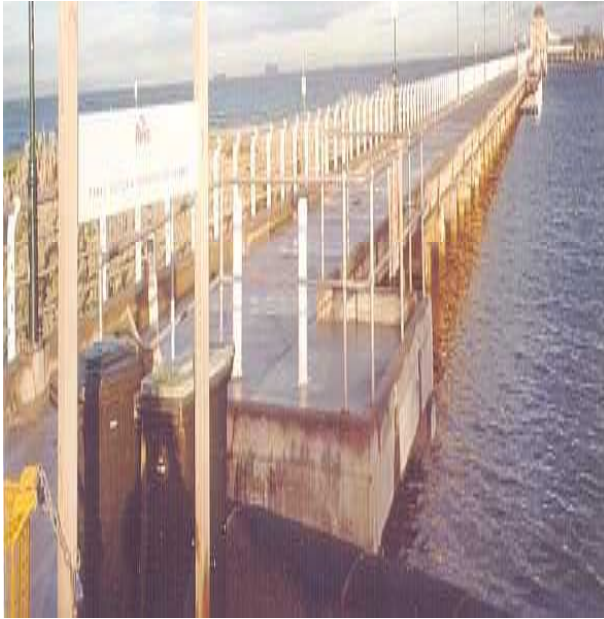
Piles have shown a high corrosion activity and many corrosion-induced cracks. The corrosion rate measured on the upper part of the pile is rated moderate to high. Some of the piles near the shoreline have large vertical cracks, which may have been initiated at the time of construction due to the hammering operations.

Damage due to corrosion activity is expected to increase if no remedial action is taken for the structure. The load carrying capacity of all the elements is adequate for the current rating. However, further steel corrosion will lead to more loss of bond between concrete and the steel as well as the reduction in effective steel areas.

Repair of cracked and delaminated concrete is recommended together with the application of an impressed current CP for these damaged elements. These and the rest of the structure would need to be monitored for 5 years before further remedial action is undertaken.

## 8. REFERENCES

- 1 BROOMFIELD, J.P., RODRIGUEZ, J., ORTEGA, L.M. and GARCIA, A.M. "Corrosion rate measurement in reinforced concrete structures by a linear polarization device". In: R.E. Weyers (ed.), Philip D Cady Symposium on corrosion of steel in concrete. *ACI Special Publication, 151-9*. 1994, 163 pp.
- 2 BUILDING RESEARCH ESTABLISHMENT. "The durability of steel in concrete : Part 2 – Diagnosis and assessment of corrosion-cracked concrete". *Digest 264*. Building Research Establishment, Garston, Watford, UK. 1982.
- 3 BROOMFIELD, J.P. "Corrosion of steel in concrete, understanding, investigation and repair". Chapman & Hall, London, UK. 1997, pp 22-25.



*Figure 1: General view of jetty*



*Figure 2: Vertical cracking in pile caused by corrosion of steel bar*



*Figure 3: Cracking and delamination in pile induced by corrosion*



*Figure 4: Cracking with rust stains appearing at lower part of crossheads*



*Figure 5: Spalling of cover concrete at crosshead soffit*



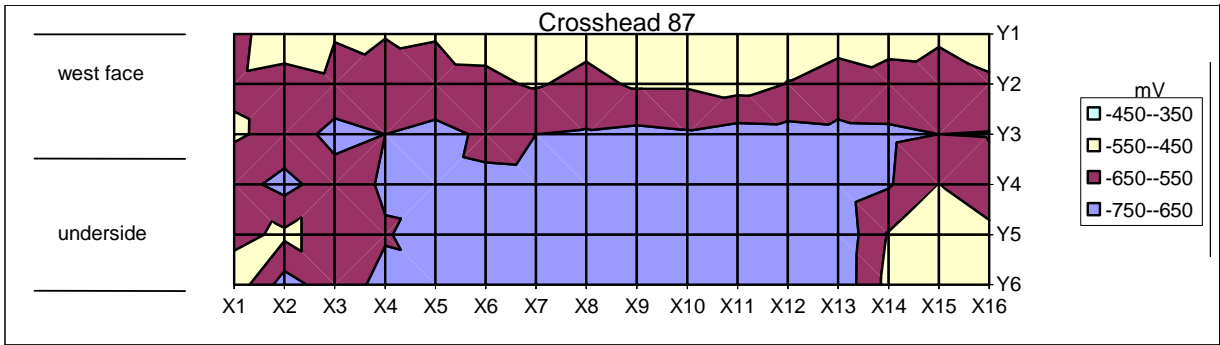
*Figure 6: Complete delamination of cover concrete at crosshead soffit*



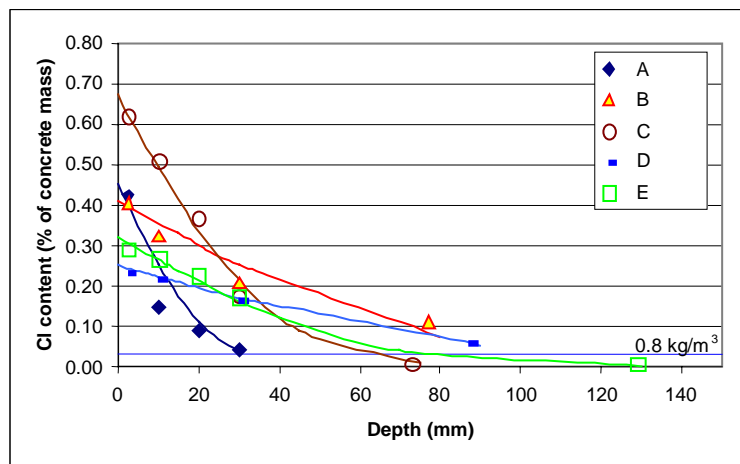
*Figure 7: Cracking along the upper reinforcing bars and honeycombing at the bottom edge of crosshead*



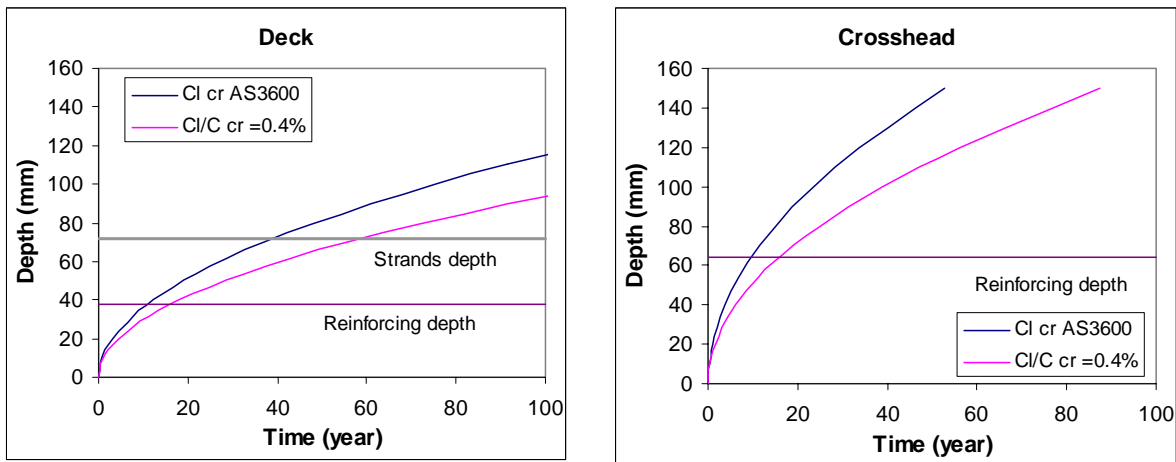
*Figure 8: Localised corrosion at the edge of deck plank Bent 77*



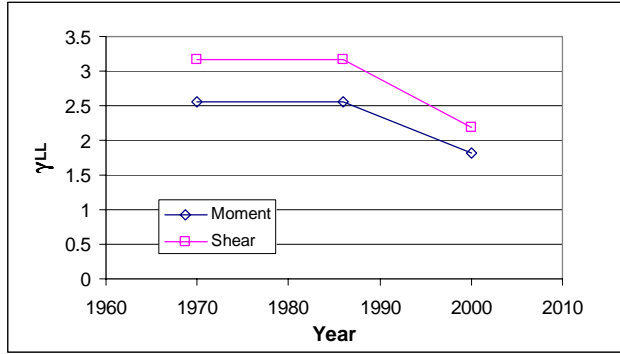
**Figure 9: Half-cell potential map showing elements tested**



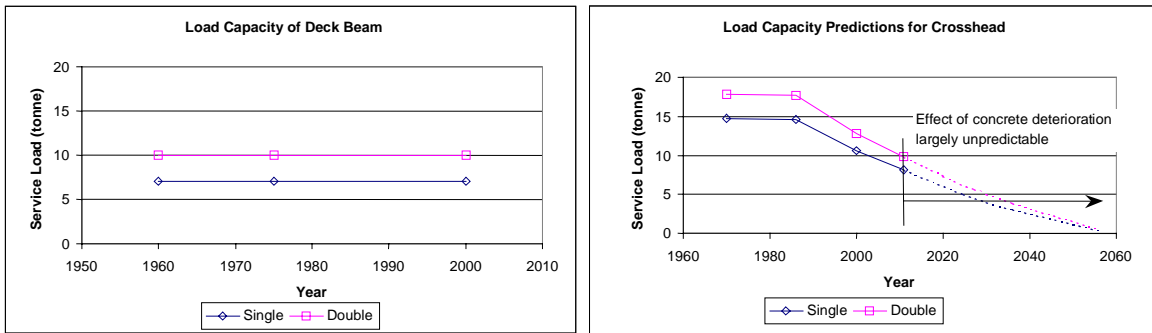
**Figure 10: Chloride profiles for the elements tested. Both experimental points and fitted curves are shown**



**Figure 11: Prediction of chloride penetration into concrete for the deck and crosshead**



**Figure 12: The live load safety factors of deteriorated crosshead under 10 tonne double axle load**



**Figure 13: The service load capacity of deck beams and crossheads**