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Long-term durability of reinforced concrete piles from the Hornibrook Highway Bridge

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Abstract:

After more than 75 years continuous exposure to the Pacific Ocean waters on the Queensland coast the 879 reinforced concrete driven piles that supported the superstructure of the Hornibrook Highway bridge appeared to be in remarkably good condition when the bridge was demolished during 2011-2. Detailed investigations revealed excellent, very hard concrete with pH values still around 12 and very little evidence of serious corrosion of the steel reinforcement. The concrete chloride content at the reinforcement was considerably more than the usually accepted limits. However, a few isolated occurrences of very severe localized reinforcement corrosion were found during demolition even though there was little visual external evidence. Possible reasons for the various observations are discussed, together with the practical implications.

Keywords: Reinforced concrete, durability, tidal-zone, chloride, alkalinity, corrosion.

1. Introduction

The management of the technical and economic risks associated with ageing infrastructure is of increasing importance for engineers, infrastructure authorities and asset owners. Typically the first concern is with the current or existing state of an item of infrastructure. Assuming the infrastructure is considered acceptable for current service, the second concern is with its likely future condition, or equivalently, its likely rate of deterioration from its current condition. Ideally this requires sound prediction capability for life-cycle assessment (Paik and Melchers 2008). Typically this involves not only the prediction of structural deterioration but also the prediction of possible increases in natural and applied loadings. Both need to be made, usually, for time horizons extending over many years.

The majority of models available for the prediction of deterioration have been derived from short-term laboratory experiments (e.g., Coronelli et al. [2009]). Where long-term investigations have been made, they rarely (if at all) replicate true in-service loading and environmental conditions (Duffo et al. [2004]; Malumbela, Alexander, and Moyo [2009]). For sound prediction of the likely future deterioration of actual structures, it is highly desirable for deterioration models to be based on actual field investigations, using detailed observations and analysis, such as the studies by Woodward and Williams (1988); Poupard et al. (2006); Melchers and Li (2009a) and Pape and Melchers (2013a, 2013b). Such studies increasingly are considered to be very important for the development of realistic models for prediction (Angst et al. [2009]). Many inspections, investigations and reviews of existing bridges are made in practice. Unfortunately, most of these are not available in the open literature or for open discussion. The number of detailed investigations of case studies that are available in the open literature is limited as noted, for example, by Gjrović (2009) and Angst et al. (2012).

The present paper reports a detailed investigation of the reinforced concrete piles that became available as a result of the demolition, during 2011-2, of the main spans of the Hornibrook Highway Bridge (Hornibrook bridge for short). The bridge was part of the Hornibrook Highway crossing at Bramble Bay, north of Brisbane, Queensland, located immediately adjacent to the Pacific Ocean. During the demolition of the bridge it was observed that the reinforced concrete piles appeared still in good condition despite their more than 75 years in service in the aggressive coastal marine environment. This led to the present investigation.

An overview of the structural aspects of the bridge is given next, followed by a short summary of previous investigations of the condition of the reinforced concrete bridge piles. Observations regarding the condition of the concrete and of the steel reinforcement at the time of demolition are then given, noting that, overall, both concrete and steel can be considered to be still in very good condition despite the age of the structure and the exposure environment. Possible reasons for this are then discussed, followed by a summary of likely direct implications for design and maintenance of marine reinforced concrete structures.

2. Background

The Hornibrook Bridge was built by M.R. Hornibrook Pty. Ltd. as a privately commissioned construction project (Stedman et al. [2006]). It consisted of 294 spans, was 2684 m long and was the longest road viaduct across water in Australia when completed in 1935 (Fig. 1). It consisted of an iron-bark timber girder and deck superstructure with a bitumen overlay, supported on timber corbels at each pier. All the 293 piers were similar and each consisted of a cast-in-situ reinforced headstock supported on three pre-cast reinforced concrete piles (Fig. 2). All the 879 piles in the bridge were driven piles, having been cast on their sides near the construction site. They were of nominal size 15 inches by 18 inches (381mm x 457mm) with 1.5 inch (37mm) 45 degree chamfers along their edges. Their sizes varied by up to about +/- 10mm from nominal. The piles contained 4 reinforcing bars of 1-1/4 inch (32mm) nominal diameter, located in the corners of the cross-sections with nominal cover of 2-1/2 inches (62.5mm). For the samples examined (see below) the cover was found to vary between 40 and 70mm.

Figure 1 here

Figure 2 here

Ownership of the bridge was transferred to the (then) Main Roads Department of Queensland in 1975. Inspection of the bridge in 1979 showed the timber deck had deteriorated sufficiently for the bridge to be closed to vehicular traffic (Derbyshire et al. [2007]). Field inspections in 2011 indicated some reinforcement corrosion in the concrete crossheads and much more in the RC approach decks. The bridge was then closed to pedestrian traffic and subsequently demolished, except for the historic entry portals and adjacent reinforced concrete beam and slab spans. During its 75 years of service the reinforced concrete in the bridge had been exposed continuously to the local coastal marine environment. The bridge piers were subject to immersion, tidal, splash and atmospheric exposure conditions, including seawater currents. Because of the shallow water depth relative to the tidal range, most of the reinforced concrete piles were fully exposed to the atmosphere during low tide. When the piles were extracted during demolition, most were covered with various forms of marine growth particularly in the immersion and tidal zones. Acid sulphate soils are known to exist in the vicinity of the bridge and these may have had some influence on durability.

3. Earlier investigations of bridge structural condition

There were at least two conditions assessments prior to demolition of the bridge. In 1988 pier 1 (only) was examined and was considered, from visual inspection, to be in good condition (Carse [1988]). Laboratory testing on cores showed concrete densities of 2342-2473 kg/m³ and compressive strengths in the range 31.6-56.7 MPa. Concrete chloride levels were reported as in the range 0.3% - 2.6% by weight of cement (1.4 - 12.2kg/m³). From phenolphthalein testing it was concluded there had been no carbonation. The report noted that the piles were Class A concrete with a design aggregate/cement (A/C) ratio = 3.5/1 while the headstocks and approach decks were Class B with A/C = 5/1. The report also noted the much superior durability performance visually observed for the pre-cast piles. This was attributed to their higher cement content.

The 1988 assessment also used concrete surface potential for the headstock of pier 1 and resistivity measurements for both the headstock and the piles. This concluded that the corrosion potentials in the headstock were high with a 95% probability that corrosion was occurring. Given the environment and the long exposure period and the high chloride contents this should be expected. The resistivity measurements gave a similar outcome for the headstock but it was concluded that 'corrosion is unlikely to occur in the piles'. These outcomes were considered consistent with the other measurements and with the visual observations.

In 2003 a second, much more extensive, condition assessment of the bridge considered the piers, from visual inspection, to be in good condition (Adams and Carse [2003]). Some piles showed hairline longitudinal cracking where they intersected at their tops with the headstocks and this was attributed to corrosion of some of the reinforcement in the headstock. Some rust staining but very little spalling of concrete cover was observed for the piles. Concrete cores were taken at the tops of 6 piles and the lower parts of 4 others. These cores showed concrete densities in the range 2344-2498 kg/m³ and compressive strengths 31-69 MPa. Concrete chloride levels and profiles were reported based on an assumed cement content of 470kg/m³. At the reinforcing bars the chloride content was reported as around 0.6-0.8% by weight of cement (3-4kg/m³) for the lower parts of the piles and in the range 1-3% by weight of cement (5-14 kg/m³) for the upper parts of the piles. There, some hairline cracking had been observed during inspection and was thought to have started in the headstocks. The chloride contents also were reported to be higher towards the tops of the piles and adjacent to the few hairline cracks that were observed. Phenolphthalein testing led to the conclusion there was no significant carbonation of any of the concrete.

Both the 1988 and the 2003 investigations showed that the concentration of chlorides in the piles much exceeded the usually accepted threshold of 0.4-0.6% (by weight of cement) above which reinforcement corrosion usually is considered to initiate (e.g. Richardson [2002]). However, as noted, the field inspections found little exterior visual evidence of serious reinforcement corrosion in the piles. Only a few hairline cracks and some minor rust stains were observed on just a few piles. There was no evidence of corrosion damage, such as cracks larger than hairline, significant rust staining at such cracks or elsewhere, spalling of concrete cover or major cracks along reinforcement bars. The cast-in-situ headstocks, on the other hand, showed some areas of severe corrosion damage to the concrete (Adams and Carse [2003]).

4. Present investigations

The present investigations were undertaken during and soon after demolition of the bridge. From the beginning of demolition, one of the authors inspected the bridge piles and pier headstocks at regular and frequent intervals as demolition progressed (Fig. 3). These elements were also inspected when they became available as pieces in the demolition contractor's yard. This showed the generally excellent condition of almost all of the piles and of many of the cross-heads, as shown by typical examples in Figs. 4 and 5. However, the reinforced concrete beams in the approach

ramps showed considerable concrete spalling and visual evidence of reinforcement corrosion (Fig. 6).

Figure 3 here

Figure 4 here

Figure 5 here

Figure 6 here

Since detailed investigation of the headstocks, approach ramps beams and slabs was precluded, it was decided to concentrate on investigation of the piles overall and in particular the sections in the tidal zone. This zone often is considered one of the most severe exposure conditions for corrosion of reinforcement in reinforced concrete structures (Richardson [2002]) as illustrated by the examples given by Gjorv (2009) and Angst et al. (2012). However there are other cases where reinforced concrete structures have lasted for long periods of time even in the tidal and immersed zones (Lau et al., [2007]; Gjorv [2009]).

For the investigation reported herein, 7 different piles were selected at random from the many available in the demolition yard. From each of these a sample approximately 2-2.5 m long, was cut, covering the region of the piles that had been exposed to the tidal zone. The pile selection included two that were considered by the demolition contractor to be less sound than most of the piles recovered during demolition.

The selected pile samples were transported by road to the structural engineering laboratory at The University of Newcastle. Each pile sample was marked for orientation and identification and for correlation with the visual inspection of the surfaces of the pile samples for surface defects, cracking and rust stains. Three of the pile samples were sawn into shorter lengths and then broken open (with considerable difficulty) using sledge and jack-hammers. This allowed the reinforcing bars to be uncovered and photographed. Concrete surfaces immediately adjacent to these bars were photographed also. In the few cases where signs of corrosion were observed on the exposed reinforcement, samples of the usually very thin corrosion product were collected for X-Ray Diffraction (XRD) analysis.

The pile samples were used to obtain smaller samples of the interior concrete by taking concrete cores (100mm diam.) in directions perpendicular to and roughly in the same plane as the longitudinal axes of the piles. This was to obtain representative samples of chloride penetration into the piles. Individual core samples were used for petrographic and XRD analyses. Samples of concrete were taken also from the exterior surfaces of the piles and also analysed by XRD to allow comparison between the interior and exterior concrete properties.

Some of the freshly cut cross-sections of the pile samples were tested for pH using a phenolphthalein spray. Immediately after a colour change was noted the sprayed surface was photographed to record the colour over the cross-section. Other freshly cut cross-sections were used to obtain more accurate pH readings at multiple locations, using a laboratory pH meter, after wetting the area with a few drops of distilled water and then measuring the resulting liquid pH.

Noting that the 2003 investigation had found very high chloride contents in randomly selected piles, an independent check on those results was made as part of the present investigation. Also, to assess whether there was a directional effect for concrete chloride content in the piles, a 100 mm diam. core was taken right through one of the pile samples and another at right angles to it. These cores

were sliced into 15 segments of 20 mm thickness close to the external faces of the pile and of about 40 mm thickness in the pile interior. The slices were kept in sequence and each tested for chloride content, using a standard test method (pulverizing the concrete, digestion in nitric acid, dissolution in distilled water and analysis with a chloride selective anode) according to BS 1881 Part 124-1988. This was done by an independent testing laboratory.

Estimates of concrete strength were obtained using two different methods. A Schmidt hammer was used to test three of the pile samples at multiple locations along their length, with testing on clean smooth surfaces in conformance with ASTM C805. In addition, several concrete cores of 100 mm diam. were taken and cut into 150 mm long specimens. These were used for compression testing, using soft sulphur for end capping.

5. Observations and test results

5.1 Concrete

Two of the pile samples showed no defects such as spalling, cracking or rust staining that could be detected with the naked eye. For the other samples, a very limited amount of minor cracking was observed at isolated locations along the pile faces, sometimes coinciding with some minor rust staining (e.g. Fig. 7). As best as could be ascertained from the observation regime during demolition, such defects were not typical for all piles, many of which showed no obvious external defects or signs of rust staining. Overall the external condition of the piles indicated that the reinforced concrete was in sound condition.

Figure 7 here

Breaking open the concrete samples for internal examination and for examining the reinforcing bars showed the concrete to be very hard, so hard that several jack-hammer bits were shattered for one particular concrete pile sample. The hardness of the concrete was confirmed by the Schmidt hammer results. These ranged from Schmidt hammer reading $R = 40$ to 58 with an average of 48.4, taken over 3 different piles, each measured over 3 faces at 4 points some 0.5m apart with 5 replicate readings at each point. Since Schmidt hammer results are correlated with compressive strength (Richardson 2002), this is equivalent, approximately, to a strength range of 50-62 MPa. Usually this is an under-estimate of the actual compressive strength. By comparison, for the 100 mm diam. concrete cores the compressive ultimate strength was found to be 66-76 MPa. These sets of results are consistent and indicate a high-strength concrete. They are considerably higher than recorded in 1988 and 2003, which suggests that, as is usual in moist conditions, the concrete continued to gain strength even later in its life and perhaps much longer than conventionally considered (e.g. Richardson [2002]). This could have been the result of the concrete piles having remained wet (with seawater) throughout their 75 year life. The high concrete strength also indicates very low gas permeability (Richardson [2002]).

The internal concrete surfaces exposed by the saw cuts across the piles, by the cores and by the compression specimens, in all cases gave visual confirmation that the concretes were in a very good condition. The concrete was very dense in appearance, with a distinct lack of voids, including around the reinforcement bars. These visual observations are consistent with the high Schmidt hammer results that can be interpreted as indicating low permeability of the concrete.

Closer inspection of the concrete surfaces revealed by cutting and by crushing of the concrete during compression testing showed isolated locations where there was evidence of a white, chalk-like material, including adjacent to the reinforcement bars. It was subjected to XRD analysis and found to be predominantly ettringite (a complex hydrated calcium aluminium sulphate), calcite and

1 aragonite (CaCO_3) with some presence of brucite ($\text{Mg}(\text{OH})_2$). The presence of ettringite has been
2 associated with a delayed reaction that damages the concrete (St John, Poole, and Sims [1998]).
3 However, there was no evidence of any such damage for the concrete surfaces examined.

4 The petrographic analyses of the samples cut from the concrete showed that the coarse aggregate
5 probably consisted of pyroxene and olivine basalt, metamorphic and igneous rock materials. The
6 fine aggregate contained quartzite, strained quartz, chert, granite, feldspars, and pyroxene basalt. It
7 also showed the presence of gel usually generated by an alkali-silicate reaction (ASR), micro-
8 cracking and peripheral separations adjacent to aggregates and voids. This indicates the possibility
9 that some degree of ASR had occurred on the surfaces of some of the fine aggregate (St John,
10 Poole, and Sims [1998]). However, in view of the lack of other evidence such as cracking or crazing
11 observable with the naked eye, this is not considered to have been a significant factor in the
12 performance of the concrete.
13

14
15 For all the samples examined by XRD analyses, the concrete matrix material showed the presence
16 of ettringite and magnesium-based compounds (eg. MgSO_4 and $\text{Mg}(\text{OH})_2$), as well as varieties of
17 calcium carbonates (calcite, aragonite, vaterite). Other compounds identified were gypsum and
18 Friedel's salt. There was a higher incidence of calcite (CaCO_3) towards the outside surfaces of the
19 piles.
20
21

22 **5.2 Concrete chloride, pH and composition**

23
24 The profiles for concrete chloride content for the two cores taken at right angles through the centre
25 of the selected pile are shown in Fig. 8 as chloride ion concentration relative to the concrete volume
26 (ppm). The general shape of these profiles is typical for concretes exposed to marine environments,
27 with high concentrations at the exterior surfaces, decreasing with distance into the concrete
28 (Richardson [2002]). They also are similar, despite their different orientation. To compare with the
29 results given in the 2003 report (Adams and Carse [2003]) would have required estimation of the
30 original cement content of the concrete. Available techniques for this generally are considered not
31 credible for very old concretes (e.g. Angst et al. [2009]). However, if it is assumed the original
32 cement content was 350 kg/m^3 , the (%bwt of cement) values shown on the right vertical axes are
33 obtained. These chloride concentrations are very high, including at the locations where the
34 reinforcing bars are located (Fig. 8). If the original cement content was lower than 350 kg/m^3 , as is
35 possible for concretes made before WW2 (Wood [1948]), the chloride content as given by weight
36 would be even higher.
37
38
39
40
41

42 Figure 8 here

43
44 The pH measurements on sample cross-sections of the piles all showed similar results. They
45 showed pH readings of around 12 over almost the entire cross-sectional surface and almost no
46 evidence of reinforcement corrosion. In some isolated cases readings of pH less than 9 were
47 obtained but only in the concrete zone just inside the exterior surfaces - typically 2-5mm into the
48 concrete. This suggests that some carbonation had occurred on the exterior faces. This was
49 supported by phenolphthalein testing of the concrete. In all cases it showed a strong pink colour
50 over all the interior portion of the surfaces of the exposed concrete cross-sections, indicating that
51 the concrete pH is in the upper end of the pH range 8.2-12.0 (St John, Poole, and Sims [1998]).
52 Fig.9 shows one of the few cases with some evidence of reinforcement corrosion inside the cross-
53 section. Most of the cross-sectional area shows a pH of around 12 but there is a small zone of low
54 pH (around 6.5) at the lower part of the cross-section. Here rust staining was observed.
55
56
57
58

59 Figure 9 here

To examine whether there was a difference in concrete composition with distance away from the exterior surfaces, small pieces of concrete were taken, at about equal intervals, across the surface of the cross-sections exposed by the saw-cuts across the pile samples. These pieces were crushed and analysed by XRD. The results showed calcium carbonate (both calcite and aragonite), gypsum and minor phases of ettringite near the pile surface, that is, where the local pH was lower than elsewhere. Further into the pile, where the pH was higher, there was a greater abundance of $\text{Ca}_6\text{Al}_2(\text{SO}_4)_3(\text{OH})_{12} \cdot 26\text{H}_2\text{O}$ (ettringite) and of $\text{Ca}_2\text{Al}(\text{OH})_6(\text{Cl},\text{OH}) \cdot 2\text{H}_2\text{O}$ (Friedel's salt). At the very outside surface of the piles, the cross-sectional samples showed the existence of a 2-3mm thick layer (Fig. 10). This was identified by XRD analysis as consisting primarily of CaCO_3 and $\text{Mg}(\text{OH})_2$. These findings are consistent with the known deposition of these compounds on alkaline surfaces in seawater (Buenfeld and Newman [1984]).

Figure 10 here

5.3 Reinforcing bars

Samples of the longitudinal reinforcing bars were selected at random from those in the pile samples in the laboratory. These were taken for independent analysis by a certified testing laboratory. Table 1 shows the typical composition obtained. Comparison with typical compositions for reinforcing bars (Gjorv [2009]) showed that the composition of the bars used in the bridge piers is consistent with steels typically used for reinforcement in reinforced concrete structures.

Table 1. Typical composition of reinforcing steel (% by weight).

	C	Mn	P	S	Si	Ni	Cr	Cu	Sn
	0.25	0.83	0.013	0.041	0.09	0.02	0.02	0.07	0.009

Others < 0.001

The stress-strain behaviour of the bars testing in tensile testing showed very similar behaviour. The bars were machined down to 12 mm diameter for testing over a gauge length of 80 mm. Fig. 11 gives an example load-elongation curve converted to nominal stress and nominal strain.

Figure 11 here

The original visual inspections of the cut ends of piles stock-piled in the demolition yard had shown remarkably little evidence of corrosion of the reinforcing bars. This was supported by the results of detailed examination of steel bars inside the pile samples brought into the laboratory, noting that these samples were from randomly selected piles. In all cases the bars broken out of the samples were very well embedded in the concrete and were very difficult to expose and to remove. There were no obvious voids around the bars. The majority of the bars were in almost 'as new' condition. Only some bars showed small areas of superficial general corrosion and minor pitting. Fig. 12 gives an example. There was very little cracking of the concrete such as often seen corners and along the line of the longitudinal reinforcing bars.

Figure 12 here

On some of the reinforcing bars uncovered in the laboratory shallow corrosion with depths less than 2mm, over areas up to about 50 x 15 mm was found after removal (always with considerable difficulty) of the concrete cover. However, these cases were few. In some cases, very small areas (of size 1-2 mm), of what appeared to be bare metal were revealed when some of the bars were extracted from the concrete. Usually, these bright steel areas appeared to have been in contact with a very thin layer of black or dark-green rust that tended to remain with the concrete. Some other

bars also showed areas of dark green rusts (Figure 13). In each case small, shallow pitting was observed under these rusts when they were removed. Both rusts oxidized quickly once exposed to the atmosphere, indicative of corrosion products formed under anoxic conditions. XRD analyses was done as quickly as possible after the rusts were observed, recognizing that some of the black and green rusts would convert to ferrous oxides. The XRD analyses found a mixture of oxides including goethite, akaganeite, lepidocrocite, magnetite, and ferric oxide, but also corrosion products typically associated with anoxic conditions (ferric chloride, hibbingite (a ferrous chloride hydroxide), hydrated ferrous chloride and the so-called Green Rusts), consistent with corrosion under anoxic conditions (Refait, Abdelmoula, and Genin [1998]; Waseda and Suzuki [2006]).

Figure 13 here

As expected, the green rust products (Fig. 13) observed for some reinforcing bars quickly turned black and then red-brown, all within a few hours as a result of the change from anoxic to oxic conditions. Some of the reinforcing bars extracted from the concrete samples were stored in a clean environment in the laboratory and then examined periodically. Within 1-2 days after extraction, small iridescent beads of fluid or sometimes gel-like appearance and orange/yellow in colour were observed (Fig. 14). These were collected, dried and analysed. Some were immediately placed in the scanning electron microscope (SEM and analysed by energy dispersive spectroscopy (EDS). This revealed a predominance of iron and chloride ions. Others were analysed by XRD and correlated with ferrous chloride and hibbingite (see above). Again, these are typical rust products for chloride environments under low oxygen to anoxic conditions (Waseda and Suzuki [2006]).

Figure 14 here

5.4 Severe reinforcement corrosion

While most of the piles showed little or no obvious external signs of reinforcement corrosion, and the cross-sections of the pile samples selected at random also showed very little evidence of reinforcement corrosion, there is at least one observation that should [raise concern](#).

In the demolition process the piles were, where possible, extracted bodily from the soil foundation material. During one such operation, the pile being extracted was observed to show a sharp bend at a point about half-way along its length. Because of this unusual behaviour the pile was examined more carefully when brought into the demolition yard. There the contractor noted a transverse crack almost right through the pile cross-section at the point of bending. No other, similar cases were observed during the whole of the demolition process.

To investigate further, the contractor had the pile bent open at the transverse crack. It showed that one of the bars had corroded away almost completely and another showed significant loss of bar cross-section. The other two bars appeared to be in excellent condition. They were flame-cut to separate the upper and lower parts of the pile. Fig. 15 shows the cross-section, the two locations of the corroded bars, the flame-cut reinforcement bars, the concrete newly cracked during extraction of the pile (right side of Fig. 15) and also part of the cracked cross-section covered with light rust staining of a watery appearance. This indicates that the latter was likely to be a crack that had been present for some considerable time. Remarkably, the two corroded reinforcing bars (at left of Fig. 15) had corroded longitudinally into the concrete, in one case leaving a central remnant and in the other a very pointed narrow remnant (Fig. 16). Something similar has been observed previously for marine corrosion of reinforcement (Melchers and Li [2009a]). Very little rust deposition or rust staining can be seen on the concrete crack on the cross-section and on the exterior of the pile. There is no evidence of corrosion or rust staining on [that part of the pile cross-section broken during](#) the demolition process (the right hand side of Fig. 15). The two remaining main reinforcement bars

appear sound, as does the stirrup. Even where the lower stirrup had been (lower part of Fig. 15) there is no sign of reinforcement corrosion in the space vacated by the stirrup. The same scenario holds at the top of Fig. 15.

Figure 15 here

Figure 16 here

A somewhat similar example was found for one of the pile samples brought to the laboratory. Apart from a small amount of external rust staining at a local hairline crack it showed no evidence to suggest significant corrosion of the reinforcing bars. However, on breaking this area open, it was found that severe localised corrosion of one of the 6mm diam. steel ligatures had occurred, leaving about 25% of the original cross-sectional area. Black and green rusts were observed on the surfaces of the steel, consistent with corrosion under anoxic conditions (Refait, Abdelmoula, and Genin [1998]). Although there was some rust staining on the exterior surface, there was very little corrosion product on the concrete, including adjacent to the crack. Both in this case and in the field example, one obvious question is what had happened to the considerable volume of steel that constituted the reinforcing bar. This issue, together with the likely mechanisms involved, is considered in the Discussion below.

5.5 Interviews

John Hornibrook, son of M.R. Hornibrook, who as a young man had worked for a time on the bridge project, was interviewed. He recollected that the Queensland Cement & Lime Company had been supplied the cement used for the bridge concrete. It used coral from Mud Island in Moreton Bay as limestone to make cement (Pearson [1990]). John recalled his father would use more cement than was generally required at the time. He also noted that the piles were driven from the sand riverbank at low tide whenever possible. This was considered to be cheaper than driving from barges and also was believed to reduce micro-cracking from tensile stresses caused by pile driving.

Two experts on quarrying in the Brisbane area, Harry Simpson and David Kershaw, suggested that during the time of construction sands and aggregates were sourced from local rivers. Most likely the sands came from the North Pine River, upstream of the construction site. This is consistent with John Hornibrook's recollection and that the sands were brought in by barge.

The aggregates seen in the piers were not rounded but angular (Figs. 12, 14) and thus unlikely to be river gravels. As noted, the petrographic analysis suggested the aggregates are a hard rock. This would have been crushed for use as concrete aggregate. Apparently, the only source of hard rock in operation at the time of the construction was Moodlu quarry north-west of Caboolture, about 40 km from the construction site. The quarry site is close to a tributary of the Caboolture River, which runs into Moreton Bay just north of the bridge site. This suggests that the aggregates used in the bridge were likely to be from the Moodlu site and that they also were brought to the bridge site by barge.

6. Discussion

Overall, the condition of the piles in the bridge can be considered remarkable in view of the more than 75 years of continuous exposure in the marine atmospheric, splash, tidal and immersion zones. There was an almost complete absence of visible corrosion damage such as longitudinal cracking and concrete cover spalling for all the piles in all the exposure zones, despite the very high chloride levels. This of particular interest when viewed against the experiences with many other reinforced concrete structures that achieve only limited lifespans in such environments, typically through revealing serious, externally visible corrosion damage (Richardson [2002]; Gjorv [2009]; Melchers

and Li [2009b]). However, as noted, very severe, quite localized corrosion was discovered in one of the piles, and less severe but still significant corrosion in one of the samples repatriated to the laboratory. This should be of considerable concern since, in both cases, there was no external evidence, such as major cracking or spalling.

In analysing the reasons for the apparently good overall condition of the piles it is noted that at the time of the construction of the Hornibrook bridge, relatively modern additives for concretes, such as fly-ash, silica fume, and others to reduce diffusion capability or to reduce cement contents, and setting agents such as calcium chloride, were not part of conventional practice. They therefore cannot have influenced its long-term performance. Also, at that time blast furnace slag cements were not used in Australia.

Gjorv (2009) in his comparison of the durability of various reinforced concrete marine structures stressed the need to have high quality concrete in order to achieve good reinforcement durability. For the Hornibrook bridge this appears to have been achieved, as evidenced by the lack of voids, high density, high concrete strength and low permeability of the concrete and its high alkalinity indicated by the pH still being around 12 after some 75 years. These all could reflect a higher cement content than was customary at the time, as suggested by John Hornibrook, but it must be remembered that the typical cement content in the 1930s was only about 15% as measured by the standard 1:2:4 (cement: fine aggregate: coarse aggregate) mixes (by volume) then used. Also, at that time there were no limitations on the addition of water except for the need to achieve a workable mix (Wood 1948). Workability of a concrete mix and good compaction are necessary for good contact between the cement slurry in the concrete and the steel reinforcement. This is an important aspect for achieving reinforcement durability (Gjorv [2009]). However, high water content is well-known to be detrimental to concrete strength (Richardson [2002]). This did not appear to have affected the Hornibrook concrete, since, as noted, it showed remarkably high strength for what was meant to be simply Class A concrete with typical minimum compressive strength 3500psi (25MPa). In turn this suggests that the reported trend for M.R. Hornibrook to use more cement than required is credible.

An important factor in achieving a high degree of contact between concrete and steel bars is the degree of compaction applied to the concrete during casting. One of the four surfaces of each pile is slightly rougher, indicating the piles were cast horizontally. Breaking open samples of the piles in the laboratory was, as noted, very difficult. It showed very few voids in the concrete mass, including immediately adjacent to the reinforcing bars. There was no evidence of any of the voids having a preferential location along the bars (such as along the bottom of bars as might be expected for poor compaction). These observations indicate that the concrete in the piles received good compaction during casting, presumably through the labour-intensive tamping technique. Mechanical vibrators were only at that time being developed in the USA (Dolen [2010]) and thus unlikely to have been used on the Hornibrook bridge. Overall, these observations indicate the high quality 'workmanship' involved and also that the concrete is of high density (and by implication low permeability) throughout.

Even with very good compaction, in practice some small amount of void space will remain between the concrete and the steel reinforcement at some locations along the bars. Typically these voids will occur under any horizontal bars. These pockets of air and moisture, although small, can cause a limited amount of early corrosion perhaps under the influence of chlorides if they have migrated there sufficiently quickly. Provided the surrounding concrete is sufficiently impermeable and there is no access of oxygen such as through a crack, this mechanism ceases relatively quickly through lack of oxygen, leaving a small amount of mainly anoxic corrosion product. This mechanism is broadly consistent with the corrosion of steels in water, including seawater when oxygen is limited or curtailed such as through the build-up of rusts (Melchers and Li [2006]). It is likely that this

mechanism is responsible for the observations of minor corrosion such as shown in Fig. 12 and the anoxic rusts shown in Fig. 13.

A high degree of contact between the concrete and the reinforcing bars also is likely to ensure that overall the pH of the steel reinforcement surfaces is similar to the pH of the mass concrete. The concrete pH of around 12 (Fig. 9) around the steel bars is much higher than the pH below which steel corrosion commences – this is about 9 for steel in water (Jones [1996]) and is slightly higher in the presence of chlorides, such as for steel in seawater (Duffo et al. [2004]). This behaviour is governed by thermodynamics (Gibbs Free Energy) and often represented in the well-known Pourbaix diagram (Jones [1996]). That corrosion does occur when the pH becomes lower than about 9 is seen in Fig. 9 but also in detailed observations that have been conducted on other older reinforced concrete structures. For example, 60 year old concretes exposed to wet marine atmospheres and periodic seawater wash-over showed no corrosion of reinforcement steel for sound concrete with $\text{pH} > 9$ but, on the same cross-section, corrosion was clearly visible where the local pH was just below about 9 (Melchers and Li [2009a]).

An important question is why the pH of the concrete piles was, on the whole, still at around pH 12, after more than 75 years. Maintenance of pH reflects a high alkalinity content of the concrete, usually attributed to cement content. It has been proposed (Sagues, Moreno and Andrade [1997]) that loss of alkalinity is caused principally by rapid leaching of the soluble alkalis KOH and NaOH and the much slower leaching (Johnston and Grove [1931]) of the almost insoluble major alkaline component, calcium hydroxide $\text{Ca}(\text{OH})_2$. Such a mechanism appears consistent with the present observations (Figs. 4, 5 and 9) that may be attributed to higher cement content and also with other observations (Melchers and Li 2006) including that corrosion initiation and corrosion damage are delayed in the presence of alkaline aggregates (Melchers and Li [2009b]). Other factors that could have contributed to alkali reduction for the piles have been considered (Pape and Melchers [2013]) but are likely to be less important.

The high pH in the concrete is also likely to be responsible for the deposition of calcite and brucite on the lower exterior surfaces of the piles, that is, the parts of the concrete piles in the immersion and in the tidal zone (Fig. 10). These compounds, which are present abundantly in natural seawater, are the principal components of seashells and thus more alkaline than seawater and are known to deposit on surfaces with high pH (Buenfeld and Newman [1984]). Typically they form a dense layer (see Fig. 10) and contribute to reducing the permeability of the exterior surfaces. A similar explanation has been proposed for the behaviour of other concrete structures, including the survival for piles in the tidal zone but not necessarily above this level (Melchers [2010]). The often-quoted explanation that corrosion of reinforcement in the immersion zone is inhibited by the low oxygen content of seawater is inconsistent with the high corrosion rates for bare steel in the same environment (Schumacher [1979]).

Consider now the very aggressive and highly localized corrosion of the cracked pile, as well as the laboratory sample with a crack, noted in Section 5.4. Such aggressive corrosion over a confined region is not a new phenomenon. It has been observed also in other older reinforced concrete structures exposed to marine environments, often similarly with hairline cracks, into the concrete, intersecting with the reinforcement, and little or no external evidence in the form of rust stains (e.g. Melchers and Li [2009a]). Conventionally hairline cracking is considered harmless if it is shallow (Beeby [1978]; Richardson [2002]) but deeper cracking, even hairline cracking, to the reinforcement or close to it provides a possible avenue for alkaline leaching, thereby causing a slow local drop in pH (Melchers and Li [2006]). This could then, eventually, allow initiation of localized corrosion. Once initiated, the localized corrosion can be expected to follow a trend not unlike corrosion of steel in seawater (Melchers and Li [2006]), Moreover, if very low localized oxygen or anoxic conditions develop where the crack intersects with a reinforcing bar, localized severe pitting

can occur. This mechanism is well-known for pitting corrosion of steel (Wranglen [1974]) and show very low pH conditions and the development of ferrous chloride within pit(s). Ferrous chloride is one of the very few soluble rust products and thus one of the few that could leach out of pits and cracks and eventually be transported by water flow. All the normal ferrous and ferric oxides and hydroxides, usually considered as ‘rusts,’ are essentially insoluble (North [1982]; Waseda and Suzuki [2006]) and thus not easily transported by fluid flow, particularly, for example, though hairline cracks. Under suitable tidal or other continuously moving fluid flow or rain exposure conditions the ferrous chloride and any oxidized rust products that may form from it would be washed or transported away, unable to settle on the exterior concrete surfaces and thus leaving no rust stains. This scenario is consistent with what was observed, both in the present cases and earlier (Melchers and Li [2009a]).

The observations from the Hornibrook bridge and the other cases mentioned show that while high quality concrete, with very low permeability, high strength, high pH of concrete and also high remaining alkalinity is required for applications in aggressive marine environments, including in the tidal zone (e.g. Gjorv [2009]), very aggressive localized corrosion can still occur. The present observations support earlier observations that the critical factor that led, eventually, to the severe localized corrosion was the presence of an imperfection such as a deep (hairline) crack into the concrete. These can be considered to facilitate the (slow) lowering of local pH, sufficient for corrosion to initiate. It follows that, in general, attention must be given in practice to what might appear to be minor concrete cracking and also to the presence of quite small rust stains. The evidence now available shows that severe localized reinforcement corrosion may be occurring without much external evidence such as the rust staining usually associated with such corrosion.

Although Fig. 15 shows a rather alarming amount of corrosion, the available evidence suggests that, for the present bridge at least, it is a rather rare occurrence, being only around 0.1% of the total number of piles, or equivalently, of the total exposed concrete surface area of all the piles. Also, no reports exist that the impaired pile caused any obvious signs of structural distress during the service life of the bridge. While superficially it might appear that potentially all hairline cracking might be implicated, a closer look at Fig. 15 suggests that the pile was cracked during construction, prior to being driven. This can be seen from the ratio of the area of rust stained concrete to area of freshly broken concrete and the effect on bending of the 4 reinforcing bars. Very little cracking in concrete structures is of this type (except perhaps for construction joints). This suggests that whole-scale examination of concrete surfaces for hairline cracking may not be useful. It also provides a reminder of the importance of redundancy and ductility in design and in the assessment of structures.

Even if detailed inspection of concrete surfaces for hairline cracking and small amounts of rust staining was considered desirable, it must be borne in mind that such signs do not necessarily correlate with corrosion damage as serious as in Fig. 15 but may be the corrosion of ties, or only the beginning of corrosion. Ideally the extent of corrosion needs to be ascertained prior to any remedial action. Unfortunately, the techniques presently available for this appear to be limited in their capacity to determine the degree of reinforcement bar corrosion loss. The present observations reinforce the need for such tools to be developed.

The small beads observed to form on the outside of some reinforcing bars after they had been removed from the concrete (Fig. 14) also may be the result of the mechanisms associated with pitting corrosion under chloride conditions. As noted above, the pit interiors have a very low pH and are known to contain (water soluble) ferrous chlorides (Wranglen 1974). When the corrosion pits are exposed to a relatively humid atmosphere, such as occurs when the bars are exhumed from the concrete, the ferrous chlorides will exude from the pit interiors and oxidize almost immediately to other rust products (Waseda and Suzuki [2006]), giving rise to the beads seen in Fig. 14. Similar

observations have been made by others (e.g. North [1982]; Neff et al. [2005]) but not in the context of reinforcement corrosion.

It has been recognized for a long time that small variations in steel composition have negligible effect on immersion corrosion, including long-term corrosion (e.g. Evans [1960]). It follows that the observations given above for the corrosion of the reinforcement for the bridge cannot be attributed to the properties of the reinforcement.

An interesting question about reinforcement corrosion for the Hornibrook bridge is the contrast between the good overall performance of the nearly 900 reinforced concrete driven piles and the somewhat poorer overall performance of the nearly 300 headstocks (Fig. 2) and the considerable corrosion damage observed on the underside of the slabs on the bridge approaches (Fig. 6). All of these concrete structures were part of the same project, under the supervision of the same engineer and constructed at much the same time. It seems reasonable to assume that the materials used, the quality of construction and of workmanship, were similar and unlikely to have caused the differences in the long-term performance of the piles, the headstocks and the approach decks. The most likely reason is the differences in concrete: Class - A for the piles and B for the rest (Carse [1988]). Class A concrete has a greater cement content than Class B and therefore greater alkalinity reserves ([assuming, as in the present case, all other factors being the same](#)). A second reason may be local exposure conditions, as is known for other situations. For example, the undersides of steel roofing often show more severe corrosion (in marine atmospheres and elsewhere) and this is usually attributed to longer periods of ‘wetness’ (Evans [1960]; Jones [1996]) rather than any difference in chloride deposition (which usually is less on undersides). A parallel situation is likely for bridge decks and to a lesser extent for the headstocks, both less exposed to drying effects than the piles. This is an obvious area for further research.

Conclusion

The following conclusions are drawn from the observations and interpretations given above:

1. The reinforced concrete piles in the Hornibrook bridge represent excellent and multiple examples of the extremely good long-term durability that can be attained in reinforced concrete structures despite very high chloride contents. The available evidence both from visual observation of the piles and from tests on random samples, shows that, overall, the concretes were well-made, showed a high degree of physical integrity, high strength, low permeability, lack of voids and high pH (mostly still around 12).
2. Where severe reinforcement corrosion was observed it could be associated with fine local cracking though to the reinforcement and sometimes also with small amounts of exterior rust staining. This was interpreted as consistent with the notion that localized alkaline leaching is an important component in contributing to the considerable localized reduction in pH of the concrete and the development there of highly localized corrosion, even in concretes with high pH elsewhere.
3. The present findings show that structural engineers should be concerned with detection of possible localized regions of highly localized corrosion rather than overall concrete chloride content of a reinforced concrete structure. Such regions of localized severe corrosion are associated with fine but deep hairline cracking or similar imperfections and cannot always be detected easily by visual inspection, particularly in tidal or splash zone locations or where rain-waters may wash away any corrosion products exuding through the cracks. [How to deal with this issue in practical bridge inspection, and in possible prevention or amelioration, are areas for further consideration and research.](#)

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This investigation has its origins in the observation (by RJH) during the early stages of the demolition of the Hornibrook bridge of the good condition of the piles. Eventually this led to agreement with (then) Department of Main Roads Queensland (now Queensland Department of Transport and Main Roads), the demolition contractor JF Hull and on-site representative Clayton Smith, to give The University of Newcastle access to pile samples and to share information. Australian Research Council funding allowed TMP and later IAC to work on the project as post-doctoral researchers, provision of specialist assistance (Petrographic International Pty. Ltd., Fred Salome of CTI, Sydney) and use of University facilities (Jenny Zobec of the EM/XRay Unit and the staff of the Civil Engineering Laboratory) to carry out much of the detailed materials investigation. Also, RJH and TMP contributed to background information and arranged discussions with John Hornibrook (the son of M.R. Hornibrook) and also with local experts Harry Simpson and David Kershaw regarding aggregate sources. The authors much acknowledge the contributions of all participants. Without them this project could not have been completed.

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Figure 1. Hornibrook Highway bridge prior to demolition. At each end the approach span and the historic portal building have been retained.

Figure 2. Underside of bridge in 2010 showing reinforced concrete piers each consisting of a headstock supported by 3 reinforced concrete piles and supporting ironbark timber corbels that in turn support ironbark timber girders and an ironbark timber deck. There is only some rust staining and no visible concrete cracking and cover spalling of the headstocks and piles.

Figure 3. General view in 2011 of the underside of the bridge, showing the generally good visual condition of the reinforced concrete piers and headstocks.

Figure 4. Typical complete pile recovered during demolition, showing typical exterior with no signs of cracking or rust staining. The far end shows part of the reinforced concrete headstock.

Figure 5. Close-up of part of one of the 293 cast-in-situ reinforced concrete headstocks recovered during demolition. There is little evidence of corrosion of the main reinforcement evident on the cut cross-section. The bar ends and the ligature have corroded since the section was exposed during demolition and storage in the demolition yard.

Figure 6. Underside of bridge approach slab and supporting beams looking North showing considerable spalling of concrete cover and clear signs of reinforcement corrosion.

Figure 7. Example of minor longitudinal and transverse concrete cracking and associated rust staining as observed on some reinforced concrete piles.

Figure 8. Concrete chloride profiles along the centre line of concrete cores showing the location of the 32mm diam. (nom) reinforcement bars with their 62.5 mm (nom) concrete cover. Core 2 is at right angles to core 1. The vertical axes at right are based on an assumed cement content of 350kg/m^3 .

Figure 9. Pile cross-section and a typical set of spot readings of concrete pH, showing most of the cross-section is at pH around 12. In this case there is a small region where pH was around 6.5 where reinforcement corrosion had occurred. The locations of the 32mm diam. (nom) reinforcing bars are shown. The ligatures were 6 mm diameter.

Figure 10. Longitudinal cross-sectional view of pile, showing typical outer surface (left) and inner concrete (right). The thin (2-3mm thick) layer at left consists mainly of CaCO_3 and $\text{Mg}(\text{OH})_2$.

Figure 11. Typical stress-strain plot for the main (30mm diam.) reinforcement. Note: $f_y = 292\text{-}296$ MPa, $f_{ult} = 466\text{-}468$ MPa, $E = 209\text{-}212$ GPa and elongation at failure = 13% (approx).

Figure 12. Reinforcing bar as revealed by breaking open concrete pile samples, showing negligible corrosion. This was typical of most of the bars in the laboratory samples (themselves from randomly selected piles).

Figure 13. Black and green rusts along a reinforcing bar that otherwise showed no significant corrosion.

Figure 14. Orange/yellow-coloured iridescent ‘wet’ beads that appeared on the exterior surface of reinforcing bars within 1-2 days after the bars being extracted from the concrete, while stored in a cool, dry environment in the laboratory.

Figure 15. Cross-section of the pile that kinked during vertical withdrawal from foundation soils during demolition, showing rust stained crack (left) and freshly broken concrete and flame-cut reinforcing bars (right). Note lack of serious corrosion on main reinforcing bars and location of ligature at right. Photograph courtesy of Clayton Smith.

Figure 16. Detail of remainder of one of the reinforcing bars at left of Fig. 15. Photograph courtesy of Clayton Smith.