SUMMARY

Concrete wharf structures representing a range of ages, approaches to durability design and levels of environmental exposure were investigated. Design and construction features were recorded, the extent and severity of visible damage caused by reinforcement corrosion evaluated, and samples of concrete collected for laboratory testing. The current condition of the concrete in the structures was then related to commonly-used durability indicators such as quality of workmanship, depth of cover concrete, chloride ion contamination, water absorption and compressive strength to provide guidelines for the long-term management of existing marine concrete structures and the durability design for new construction.

1.0 INTRODUCTION

The investigation reported here represents the first stage in long-term research plans introduced at the 2000 NZCS conference [1]. This long-term research seeks to enable owners and managers of concrete infrastructure to make decisions about construction and maintenance that are cost effective over the life of individual structures, thereby better meeting the expectations of both operators and investors.

Marine structures were chosen as the subject of this first stage of research because reinforcement corrosion caused by chloride ingress represents the most significant concrete durability problem in New Zealand, and solutions for the structures at greatest risk will also be relevant to those in less corrosive environments. “Durability” in this paper refers to resistance to reinforcement corrosion.

One aim of the investigation is to enable asset owners to utilise site evidence to refine the maintenance and expenditure planning of existing structures.

The other aim is to enable the industry to build more durable structures in the future. To do this designers have to know how well existing materials and practice perform. Some marine structures deteriorate soon after construction while others provide a long, maintenance-free service life. In this investigation we measured design and construction parameters on wharves in good and poor condition. We then compared the measured values with those specified by clause 5.11 in NZS 3101:1995 to find out whether the NZS 3101 requirements are sufficient for a 50 year design life if met and how they might be improved.

2.0 METHODOLOGY AND RESULTS

Eleven wharves from five ports were included in the investigation. Some had been previously evaluated for the owner and others were examined specifically for this research so not all were examined to the same level of detail. They represented a range of ages, designs, and exposure conditions (see 2.1.1 below) as shown in Table 1. Only the superstructures (decks and beams) were examined.

<table>
<thead>
<tr>
<th>Wharf</th>
<th>Date</th>
<th>Structure</th>
<th>Port</th>
<th>Macro exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1937</td>
<td>Breast</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>B</td>
<td>1920</td>
<td>Breast</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>C</td>
<td>1972</td>
<td>Breast</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>D</td>
<td>1925</td>
<td>Breast (85%) Finger (15%)</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>E</td>
<td>1977</td>
<td>Wharf head</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>F</td>
<td>1977</td>
<td>Finger</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>G</td>
<td>1957</td>
<td>Finger</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>H</td>
<td>1970</td>
<td>Breast</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>I</td>
<td>1972</td>
<td>Finger</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>J</td>
<td>1920s</td>
<td>Finger and wharf head</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>K</td>
<td>1975</td>
<td>Breast</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>

2.1 Effect of environment, design and construction on wharf condition

Initially we assessed the effect of environmental, design and construction features on wharf condition.

2.1.1 Exposure

Although all the wharves were clearly in the NZS 3101:1995 exposure zone C, some were obviously more sheltered than others because of effects

1 Central Laboratories, Opus International Consultants Ltd.
such as relationship to the shore, surrounding landforms and the prevailing winds. For example a finger-type structure facing upwind into a large natural bay is more exposed than a breast structure in a small, enclosed area of harbour. Five macro-exposure zones were therefore defined to distinguish the exposure of each structure to local wind and wave conditions:

1 – sheltered
2 - partly sheltered
3 - partly exposed
4 – exposed

As shown in Table 1, most wharves were rated 3 or greater, reflecting the generally windy conditions affecting New Zealand’s coastline and the exposed nature of the ports in the sample. Nevertheless, a wide range of conditions can be experienced by structures in different parts of the same harbour as shown by the range of macro zones represented by the wharves in port 1.

Micro exposure zones were defined to distinguish the exposure of individual elements to splashing by seawater or spray. Three zones were defined:

Splash - regularly splashed in normal conditions;
Atmospheric/splash - splashed in severe storms;
Atmospheric - rarely or never splashed.

Micro exposure of individual elements was influenced by the macro exposure of the structure, and features such as:

• height above water level (recent designs employing heavy deck slabs rather than separate beams give greater height and protection);
• the nature of the backwall in wharf breasts (riprap reduced splashing by dissipating wave energy);
• the position of the element (elements facing the prevailing wind at the wharf edge were exposed to more splash);
• the shape and orientation of the elements (round/octagonal piles as used in recent design generated less splash, and beams perpendicular to the prevailing wind are exposed to more splash and generate splash themselves if close to water level);
• the amount of wash generated by vessels using the berth.

The tidal range might be expected to influence exposure, with more exposure to splash in a given time period when tidal movement is small, but this effect was difficult to gauge.

2.1.2 Design/construction issues
Not only concrete composition, but also design and workmanship contribute to ultimate durability. First, the contribution of individual design and workmanship features to the incidence of reinforcement corrosion on wharf elements was assessed by describing observations according to a 3-point rating system:

1 - no current or likely future impact;
2 - possible future impact but no current damage;
3 - has caused rebar corrosion.

The features considered were:

• assembly/alignment of elements (only three wharves affected, two by poorly aligned piles compromising cover depths in beams and one by impact damage to prestressed deck planks);
• structural and shrinkage cracking (six wharves affected, cause of cracking did not significantly influence the impact on durability);
• concrete consolidation (four wharves affected by poor compaction at the base of beams, two of which were built in the 1970s);
• drainage (deck soffits on seven wharves affected by outflow from drainage ports cast during or cut after construction);
• formwork defects (only one wharf affected, where formwork had left a porous surface finish);
• visible evidence of inadequate cover over reinforcement (all but two wharves affected, one built in 1937 and one in 1977; all types of element affected).

Next, the overall impact of all design/workmanship inadequacies on the condition of the wharf was assessed using a rating system from 1 (no impact) to 5 (major impact). All wharves except one were rated 2 or 3, indicating that the design/construction related deterioration was usually localised and had little impact on the overall wharf condition. Date of construction also had little effect.

2.1.3 Wharf condition
The extent of deterioration was assessed by calculating the proportion of affected superstructure elements in a representative area of the structure.

The overall condition of the wharf was expressed as a rating from 1 (excellent) to 5 (poor) that included the extent of damage, its severity and the impact on the wharf performance. A rating of 3 indicates repair is required. This allowed differentiation between widespread minor damage and localised severe damage.

All the wharves were affected by rebar corrosion to some degree, irrespective of age. The design/construction issues described in 2.1.2 had relatively little influence. Overall condition rating was largely determined by the extent of deterioration. Both beams and deck soffits were affected. Macro (and micro) exposure was the
major influences on wharf and element condition. Figures 1, 2 and 3 illustrate these effects.

2.2 Effect of concrete quality on durability
After assessing the effect of external features on overall wharf condition we examined how concrete composition and exposure conditions had contributed to the deterioration of individual elements. Cover depths on all elements sampled were measured with a covermeter.

2.2.1 Concrete composition
Compressive strength, cement content and volume of permeable voids were chosen to describe concrete quality.

Compressive strength and cement content are currently used as tools to specify concrete quality for durability. The data collected should thus provide a measure of the success of current specifications even though the structures examined were built to other requirements.

Compressive strength was measured on core samples. It was assumed that continued cement hydration during the 25 years or more since construction would make up for the fact that cores record lower strengths than cylinders of the same concrete, and therefore that the core strengths would approximate the original 28-day specified strengths. Cement content, based on calcium content, was measured on powdered concrete samples. Compressive strength and cement content were closely related (figure 4). Only two samples had combinations of compressive strength, cement content and cover depths that meet the NZS 3101:1995 minimum requirements for C zone concrete.

VicRoads has developed the measurement of the volume of permeable voids, “VPV”, as a quality control test for in-situ concrete because it encompasses the effects of mix design, placement and curing [2]. It is an alternative to sorptivity tests [3]. The data collected should indicate how VPV relates to actual concrete durability, and thus whether it would be a suitable parameter for performance-based specification. VPV was measured on core samples by the method described in ASTM C642. Saturated concrete densities were also calculated from the results.

VPV results represented “bad”, “marginal”, “normal” and “good” according to VicRoads’ durability classification scheme [2]. No “excellent” results were recorded. VPV correlated only broadly with cement content and compressive strength, but better with density. This indicates
that it does reflect the overall concrete porosity, encompassing not only mix design factors but also compaction and curing.

2.2.2 Ability of the concrete to protect reinforcement
Carbonation depth and chloride ion contamination at various depths from the surface were measured and related to the depth of concrete cover to determine whether the concrete was still in a condition that would inhibit rebar corrosion.

Carbonation depths ranged from zero to 46mm. Carbonation depths were lower in the splash micro exposure zone, as shown in table 2. Carbonation posed no foreseeable risk to the reinforcement, except possibly on one element in the atmospheric zone where carbonation was within 10mm of the reinforcement. The commonly accepted “square root” relationship between carbonation depth and time indicated that on this element the carbonate front would not reach the steel for another 30 years or more. Carbonation correlated broadly with cement content, compressive strength, VPV and density, but the relationships were not particularly strong.

Table 2. Carbonation depths

<table>
<thead>
<tr>
<th>Carbonation depth (mm)</th>
<th>% of sample sites in each micro exposure zone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Atm (13 sites)</td>
</tr>
<tr>
<td>&lt;10</td>
<td>46</td>
</tr>
<tr>
<td>10-19</td>
<td>31</td>
</tr>
<tr>
<td>&gt;20</td>
<td>23</td>
</tr>
</tbody>
</table>

Chloride ion concentrations were measured in samples representing 25mm bands from the concrete surface to a maximum depth of 125mm at 46 sample sites. The UK Concrete Society [4] indicates that there is some risk of corrosion at 0.05% by weight of concrete, and a high risk at 0.15%. In the band containing the outermost reinforcement, 37% of sample sites recorded less than 0.05% chlorides and 28% recorded more than 0.15%.

Chloride contents generally decreased with increasing depth, but at seven sample sites chloride levels were raised near the carbonation front. This feature was observed by Tuutti [5], who related it to the release of bound chlorides when chloroaluminates break down because of the reduced alkalinity of carbonated concrete.

The data in table 3 show that chloride contamination posed a greater risk of corrosion with increasing exposure of the element to splash.

Table 3. Chloride contents at depth of rebar

<table>
<thead>
<tr>
<th>Chloride content (% of concrete weight)</th>
<th>% of sample sites in each micro exposure zone</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Atm (14 sites)</td>
</tr>
<tr>
<td>&lt;0.05</td>
<td>86</td>
</tr>
<tr>
<td>0.05 – 0.15</td>
<td>7</td>
</tr>
<tr>
<td>&gt; 0.15</td>
<td>7</td>
</tr>
</tbody>
</table>

Within the splash zone and atmospheric zones chloride contents were related to cement content, density and VPV, and in the atmospheric zone there was also a relationship with compressive strength. When the results from all zones were pooled the only clear relationship was between chlorides and concrete density.

2.2.3 Concrete durability
The actual severity and extent of corrosion on the individual elements sampled was evaluated by the same condition rating system used to describe wharf condition (section 2.1.3).

The influence of concrete composition, carbonation depth and chloride contamination on the observed concrete durability was examined by plotting x-y graphs of various combinations of factors. A more rigorous analysis was considered inappropriate because of the inherent inaccuracies in the sampling and measurements and because different combinations of data were collected from different wharves.

Only 11 sample sites provided enough data to relate durability to NZS 3101 requirements. The two elements that met these requirements were both in the splash zone, built in the early 1970s and contained more than 350kg/m³ of cement, yet one was in good condition and one had deteriorated to a degree where repair was necessary (table 4; complying data highlighted). Thus compliance with the NZS 3101:1995 minimum cover and strength requirements for zone C does not guarantee 50 repair-free years.

Table 4. Relationship between condition and NZS 3101 requirements (complying data in bold print).

<table>
<thead>
<tr>
<th>Condition Rating</th>
<th>Age (years)</th>
<th>Comp. strength (MPa)</th>
<th>Cement content (kg/m³)</th>
<th>Cover (mm)</th>
<th>Micro exp</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30</td>
<td>44.5</td>
<td>310</td>
<td>32</td>
<td>Atm</td>
</tr>
<tr>
<td>1</td>
<td>32</td>
<td>80</td>
<td>380</td>
<td>55</td>
<td>Splash</td>
</tr>
<tr>
<td>1</td>
<td>45</td>
<td>37</td>
<td>360</td>
<td>40</td>
<td>Splash</td>
</tr>
<tr>
<td>1</td>
<td>65</td>
<td>49.5</td>
<td>350</td>
<td>53</td>
<td>Atm</td>
</tr>
<tr>
<td>1</td>
<td>65</td>
<td>49.5</td>
<td>370</td>
<td>64</td>
<td>Atm</td>
</tr>
<tr>
<td>1</td>
<td>82</td>
<td>17</td>
<td>230</td>
<td>47</td>
<td>Atm</td>
</tr>
<tr>
<td>2</td>
<td>77</td>
<td>28.5</td>
<td>310</td>
<td>69</td>
<td>Sp/Atm</td>
</tr>
<tr>
<td>3</td>
<td>32</td>
<td>71</td>
<td>380</td>
<td>60</td>
<td>Splash</td>
</tr>
<tr>
<td>3</td>
<td>45</td>
<td>49.5</td>
<td>330</td>
<td>48</td>
<td>Splash</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>51.5</td>
<td>320</td>
<td>61</td>
<td>Splash</td>
</tr>
<tr>
<td>5</td>
<td>77</td>
<td>38.5</td>
<td>320</td>
<td>66</td>
<td>Splash</td>
</tr>
</tbody>
</table>
Exposure to splash is a major determining factor of durability, as shown in figure 5.

**Figure 5. Effect of exposure on element condition (the legend indicates element condition)**

As this suggests, the condition of the elements was closely related to the level of chloride contamination at the reinforcement, as shown in figure 6.

**Figure 6. Element condition vs chloride contamination at rebar**

Within each exposure zone, concrete condition was broadly related to cement content. Correlation with compressive strength, VPV and density was less distinct or absent. Chloride content was related to cement content, compressive strength, density and VPV.

### 2.3 Service Life Predictions

Service life is often defined as the time to corrosion initiation based on the rate of chloride ingress by diffusion according to Fick’s Law. Times to corrosion initiation were calculated and compared to the observed concrete durability to ascertain how appropriate such predictions are in each of the micro exposure zones, and therefore whether they could be used to forecast the onset of corrosion in existing structures from chloride ingress data.

Figure 7 shows that for elements in the splash and atmospheric/splash micro exposure zones remaining life thus predicted (i.e. the difference between age at corrosion initiation and age at the time of analysis) does broadly correspond to observed condition. Ratings of “1” at sites where the remaining life was negative (i.e. predicted time to corrosion already exceeded) could indicate corrosion had started without having yet caused visible damage. Only one site recorded damage before the end of its predicted remaining life.

**Figure 7. Element condition vs predicted remaining life**

In the atmospheric zone, no damage was recorded even for remaining life predictions of –60 and –20 years. This suggests that where there is little exposure to splash, ingress of chloride by diffusion is not a predominant cause of deterioration.

### 3.0 DISCUSSION

#### 3.1 Implications for Asset Management

The key finding from this work is that exposure, not age, is the primary factor that determined the condition of the wharf structures investigated. Significant deterioration was evident on parts of all four wharves built in the 1970s, with major repair already having been carried out, sometimes within 15 years of construction. In contrast, three wharves more than 40 years old still have elements in good condition.

Relatively accurate predictions of time to corrosion can be obtained by the approach taken in this research:

- Define the exposure conditions for appropriate parts of each structure using the macro and micro exposure conditions identified in this research to account for climatic, design and construction features (see section 2.1.1);
- Regularly inspect structures visually to detect cracking and spalling, using a rating system to record condition and to monitor deterioration with time;
- Carry out less frequent detailed investigations (perhaps every 10 years) to detect imminent corrosion damage by checking for delamination.
and measuring chloride ion ingress and carbonation depths;
• For elements exposed to splash, apply diffusion-based models to predict time to corrosion from the measured chloride contents;
• For elements with minimal chloride contamination, estimate from the square root relationship when carbonation front will reach the reinforcement.

This approach will provide a relatively accurate method of predicting in advance the onset and rate of deterioration. Maintenance expenditure can then be planned in advance of need, giving more options, particularly for structures in severe exposure conditions.

The appropriate remedial (or preventive) treatment will be determined by the strategic importance of the particular structure: structural loading, frequency of use, revenue generated by the berth facility. It will also be determined by the likely rate of deterioration, which in turn will be determined by the exposure conditions. For example, cathodic protection is likely to be a suitable remedial option for critical structures in severe exposure conditions, while surface treatments may suffice for less important structures and/or those in more sheltered environments.

3.2 Implications for Durability Design

Durability design is based on the definition of specification criteria that will achieve a desired service life in the anticipated service environment.

3.2.1 Defining exposure conditions

Current approaches to durability design usually assume all wharf structures are in the C zone and experience similar exposure conditions. The findings of this research indicate otherwise, and therefore it may be possible to improve overall durability by designing for the range of conditions that will be experienced by different parts of a structure on any given site (refer NZS 3101:1995 clauses 5.4.1.2 and 5.4.1.4). The poor performance of one of the elements complying with NZS 3101:1995 requirements suggests that concrete quality/cover requirements should be increased for highly exposed elements, rather than being more lenient for relatively sheltered ones. Additional methods of protection, such as corrosion monitoring, cathodic protection and alternative reinforcing materials can be considered for the most extreme exposure conditions, while surface treatments may provide significant benefit where there is less exposure to splash.

All concretes examined during this research are assumed to contain type GP cement with no other binder. Marine structures built now and in the future will probably all contain supplementary cementitious materials, which should improve concrete durability. Nevertheless, there could still be benefit in providing extra protection for elements regularly exposed to splash.

Exposure conditions themselves can be controlled to some extent by design features such as maximising the height above water level, minimising the use of beams, and by providing round or octagonal piles, seawall structures that reduce splash and wave-reducing fendering systems.

3.2.2 Defining service life

A major part of successful durability design is careful definition of the concept of “design life” or “service life”. NZS 3101:1995 is based on the New Zealand Building Code (NZBC) 50-year specified intended life, defined as the time to significant maintenance.

A survey of New Zealand infrastructure owners (including three port companies) found that 90% of owners expected to carry out minor or localised repair during a structure’s life and 24% expected to carry out major repair, although 57% believed repair should not be necessary [5]. Therefore it seems that owners are prepared to accept some deterioration within the designed service life, although the same study found that 98% expected to be able to extend predictions of service life past the original design as a structure ages. Owners are therefore likely to accept that a given design life is not necessarily maintenance-free, and their acceptance is likely to increase if the time to maintenance intervention can be predicted accurately.

Service life predicted from chloride diffusion models is the time to corrosion initiation rather than the time to repair and is therefore shorter than the NZBC specified intended life. The good agreement found here between results of such predictions, observation of concrete condition and knowledge of when repair was first carried out, suggests that the predictions are reasonably accurate. In contrast, corrosion rate, and hence time to repair, is more difficult to predict. In addition, damage at the time of corrosion initiation should be minimal. Use of time to initiation as the basis for durability design of marine structures therefore presents a lower risk than use of time to repair. It may or may not deliver a specified intended life as defined by the NZBC. To avoid misunderstanding, the designer should define the basis of service life predictions used in any contract.

3.2.3 Defining specification criteria

Most of the elements investigated in detail do not meet the combined durability requirements of NZS 3101:1995. Observation, knowledge of when repair was first carried out and diffusion-based
predictions of time to corrosion all indicate that in the splash zone at least, corrosion was initiated well within 50 years. What would the time to corrosion have been if the durability design had met NZS 3101:1995 requirements?

The two elements that complied with the NZS 3101:1995 requirements are both about 30 years old and in the splash micro exposure zone. One had a condition rating of 1 and the other was rated 3. Another 30-year-old element slightly below the requirements was rated 5. Service life predictions indicated corrosion initiation within 5 years of construction for the two rated 3 and 5, and within 30 years for the one rated 1. This suggests that the NZS 3101 provisions will not give a maintenance-free 50 year life.

Unfortunately insufficient data was collected to develop better criteria as part of this investigation, although the following three trends that were detected may assist.

The success of supplementary cementitious materials in improving marine durability has emphasised the importance of binder composition and quantity rather than compressive strength. The results from this investigation show that cement content has a stronger influence on durability than compressive strength even for Portland cement binders. This means close attention should be paid to binder content.

Durability design often employs service life predictions based on laboratory measurement of chloride ion diffusion through a saturated specimen. The effective chloride diffusion coefficients obtained from site concretes in this investigation were better than would be expected from the same concrete tested by the laboratory procedure. This indicates that the laboratory test represents the tidal environment rather than the splash micro exposure zone, and so the actual time to corrosion initiation in the splash zone could be longer than predicted by such an approach. Laboratory chloride diffusion tests therefore provide a useful, low risk design tool.

The good relationship between chloride ingress and VPV in the atmospheric and splash zones (figure 8) suggests that VPV could be used as a performance criterion for acceptance of in-situ concrete, to monitor the quality of compaction and curing. Sorptivity tests better reflect the actual mechanism of moisture ingress, but VPV could provide a practical alternative in the present absence of well-developed acceptance criteria for sorptivity.

Figure 8. Chloride contamination vs VPV

4.0 CONCLUSIONS

Different parts of a single wharf structure can be exposed to significantly different environments. The specific exposure conditions experienced by a concrete element will be the major influence on its durability.

Knowledge of the exposure conditions experienced by individual components, and regular inspection to ascertain their condition will enable wharf owners to accurately forecast the onset of corrosion and develop appropriate preventive or remedial treatments.

Predictions of time to corrosion initiation based on the diffusion of chloride ions are relatively accurate, particularly for elements exposed to splash.

Durability design can be optimised by identifying the exposure conditions that will be experienced on different parts of a wharf. Splash can then be reduced by alternative design where appropriate, and/or increased protection provided for elements most exposed to splash.

The relationship of “service life” to time to corrosion initiation and time to first repair must be defined when designing a marine structure, and the owner must understand the implications. Time to corrosion initiation is recommended as a low risk service life for design purposes because it can be calculated accurately and the likelihood of damage...
at the end of life is less than for design based on
time to repair.

NZS 3101:1995 requirements will not necessarily
provide a 50-year maintenance-free life in the
splash zone.

More attention to cement (binder) content and use
of laboratory chloride diffusion data will improve
durability design.

Measurement of the volume of permeable voids
(VPV) could provide a means of specifying in-situ
concrete quality to provide assurance that
satisfactory compaction and curing are achieved.

5. ACKNOWLEDGEMENTS

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to thank the owners of the structures examined for
their interest and support.

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