# Strategic Management and Rehabilitation of an Aging Road Bridge Asset Portfolio

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## ABSTRACT

The City of Gold Coast (City) is the second largest local government in Australia, responsible for the management of multiple asset portfolios, inclusive of transport structures. With the overwhelming majority of development occurring in the past 60 years, many bridges are reaching or have exceeded their intended design life. This poses numerous challenges for the City in effectively managing its 700 bridges and major culverts, many of which are located in aggressive coastal environments or within tidal waterways. Identification of bridges in need of repair, correct scoping of the engineering investigations (Level 3) to determine the most effective rehabilitation strategies, and quality control to ensure the desired extension of life are just a few examples of such challenges.

This paper discusses how the City is managing its extensive bridge portfolio and delivery of rehabilitation projects, providing insight from the perspective of both client and consultant. Several case studies are reviewed, analysing the effective aspects of each project and discussing lessons learnt to enable continual improvement. Focus is given to methods in accurate identification of the deterioration mechanisms in defective bridge components, and the importance of specifying appropriate repair techniques to meet the desired life extension.

## 1 INTRODUCTION

The City of Gold Coast is the second largest local government in Australia based on its asset value and residential population. From the 1950's through to the 1980's was the peak development period for the canal estates on the Gold Coast. The bridges built during this time are now reaching or exceeding their intended service life. Coupled with aggressive exposure conditions and inherent construction quality issues, the effective management of these structures presents numerous challenges, some of which are listed below:

- Identification of the assets in need of further investigation and/or repair.
- Preliminary assessment of the severity and urgency of repairs.
- Project prioritisation and capital program development.
- Scoping of the engineering investigations (Level 3) in order to accurately identify deterioration mechanisms and extent of defective bridge components.
- Determination of suitable rehabilitation strategies considering cost, constructability and durability.
- Resourcing considering the highly specialised nature of works.
- Construction quality control to ensure the desired extension of remaining life is achieved.
- Budget availability taking into account rapidly increasing industry costs associated with rehabilitation of the existing structures.

Additionally, the legacy of inadequate design code provisions for durability at the time along with variable construction quality on historical projects is believed to be a major factor contributing to the premature asset deterioration and early intervention.

Although the transport structures portfolio consists of over 700 structures, this paper focuses on the rehabilitation of concrete road bridges as these represent the majority of high importance assets.

### 1.1 Analysis of the City's Road Bridge Stock

The current bridge stock includes 197 road bridges in total, out of which 191 are concrete structures. The majority are situated amongst canal estates, close to the coastline and in marine environments

(refer **Table 1** and **Figure 1** Location Map). The earliest concrete bridges date back to 1950's and are approaching 70 years old.

**Table 1: Bridge Stock Location Categories**

| Bridge Type | Over salt water | Over fresh water | Other |
|---|---|---|---|
| Road Bridge | 144 | 44 | 9 |

Over the past six (6) years the City has undertaken major capital renewal works to 18 concrete road bridges. Most of these bridges were constructed prior to 1980 and comprise 14% of the total bridge stock (refer to **Figure 2** Road Bridge Load Categories). Design standards at the time had a less considered approach to concrete durability resulting in lower concrete covers and strengths, and inferior quality concrete mixes used in construction given our present knowledge. These issues alone have led to early maintenance interventions and major renewal works to relatively young structures (i.e. less than 40 years old). Chirgwin *et al* (2009) reported similar issues on a study of RMS's bridge stock. Interestingly bridges constructed between 1971-1990 era exhibited the lowest performance and appeared to be corroding faster than their older counterparts.

With this growing understanding there has been significant development in bridge code durability provisions in marine environments (AS5100-2017) in addition to industry guidance on durability planning for Road Structures (including the recently published Z7 "Concrete Durability Series" by the Concrete Institute of Australia). It is therefore expected the newer structures will perform considerably better and achieve their intended service life of 100 years or greater.

A further 22 road bridges are currently scheduled for rehabilitation works over the next four (4) years. Based on the projects completed to date, the most common defects leading to a maintenance intervention include:

- Pile cracking in tidal zones and down to bed levels.
- Headstock cracking and spalling.
- Steel traffic barrier corrosion.
- Concrete traffic barrier cracking and spalling.
- Relieving slab settlement.
- Expansion joint deterioration.
- Abutment protection undermining and scouring.

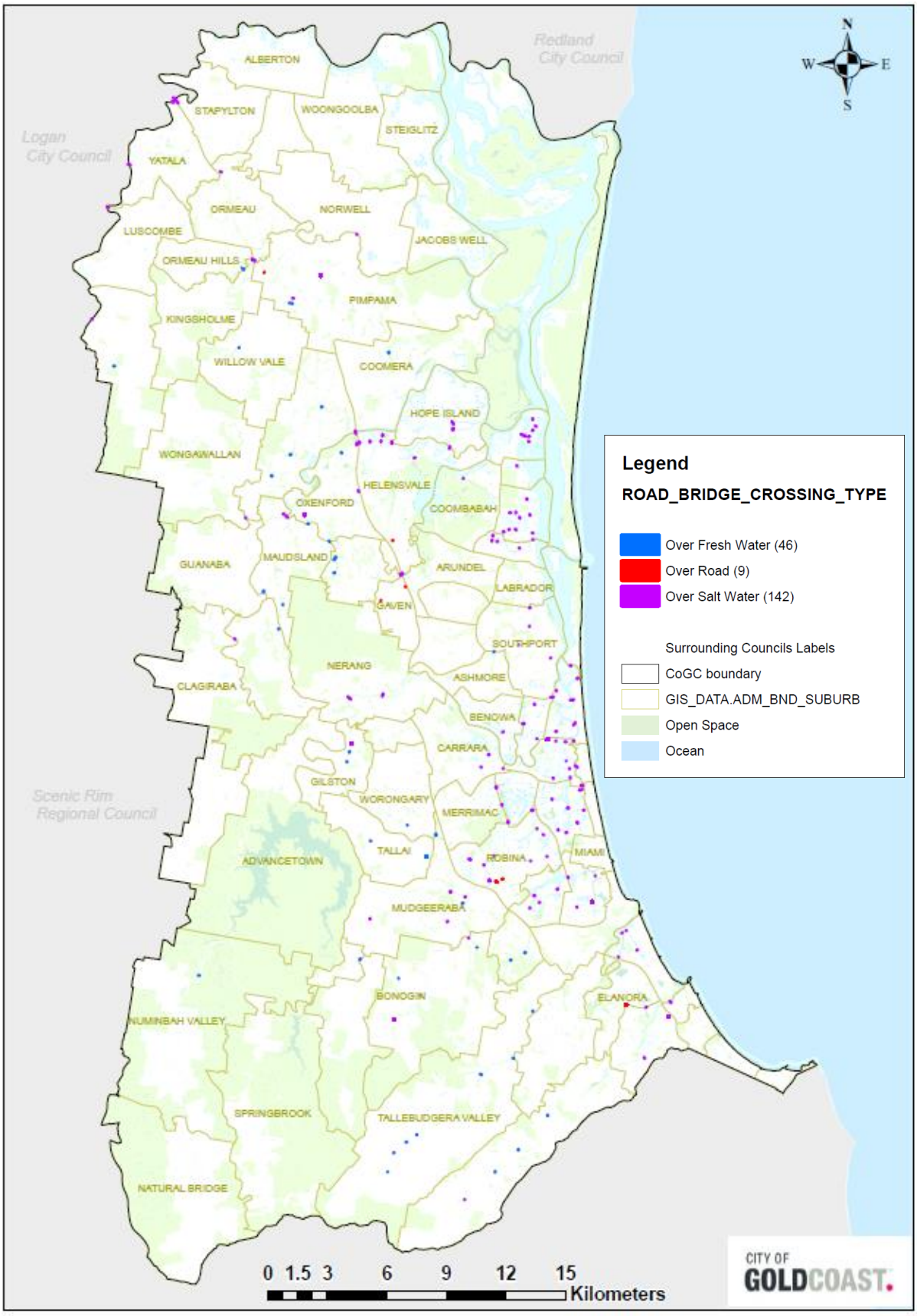


**Figure 1:** Road Bridge Location Map

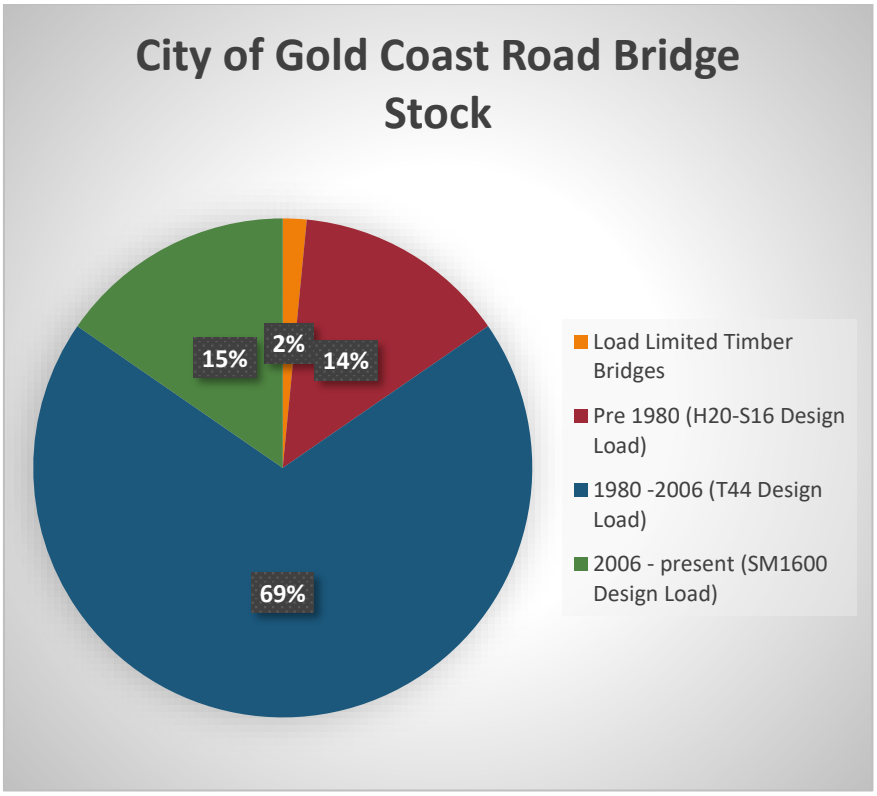


**Figure 2:** Road Bridge Load Categories

## 2 ASSET PORTFOLIO MANAGEMENT

### 2.1 Asset Management Plans

The fundamental principle of infrastructure asset management is to intervene at strategic points during an asset life cycle to maintain or extend its expected service life and maintain its performance. This is achieved by scheduling maintenance, renewal and reconstruction works at the most optimal times during the asset's life to minimise the lifecycle cost.

In order to achieve the organisational vision, comply with legislative requirements and ensure financial sustainability, the City undertakes short and long-term infrastructure planning through provision of the Total Asset Management Plan and other associated documents such as the Annual Operating Plan, Annual Budget and Long Term Financial Forecast.

Underpinning the Total Asset Management Plan, are detailed Asset Management Plans (AMP) prepared for the specific asset groups. Individual AMPs are further substantiated by operational documents which include Capital Works Programs, Maintenance Specifications, Condition Assessment Plans and Risk Registers.

Road bridges form part of the Transport Structures AMP, which also includes pedestrian bridges, boardwalks, major culverts and retaining walls. Management and rehabilitation strategies for the other asset types are outside the scope of this paper. The paper focuses on the end of life, condition driven capital renewals of existing concrete bridges (rehabilitation). It excludes level of service improvements or demand driven initiatives (upgrades).

The most common renewal strategy adopted by the City is "like for like" renewal either through rehabilitation of the individual bridge components or through replacement of the entire structure with a modern equivalent to current engineering standards. While component rehabilitation is generally a preferred option, opportunities for improvement are always considered. The final decision on the most advantageous strategy is based on the whole of life costs, budget constraints, service level requirements, safety, inherent risk, proposed future developments (growth) and an overall best outcome to the community. This is further described in Section 4.4 Strategy Selection and the case studies in Section 5.

The City employs a risk based approach to prioritising renewals, where risk is calculated based on the asset condition (likelihood of failure) and its profile (consequence of failure).

Asset condition ratings are obtained via scheduled condition assessments (Level 2). Structures in an overall condition CS4 (very poor) or bridges with principal components in CS4 are prioritised for repairs ahead of the assets which are in better condition. Presently the City has no bridges in CS5 (unsafe).

Asset profile is derived from the assessment of criticality of each bridge in the context of a road network, and consideration of other factors which put the asset in a higher priority (risk) category. The asset profiling methodology is described in Section 2.3.

### 2.2 Conditions Assessments – Level 2

The City undertakes condition assessments in accordance with industry best practice guidelines provided in the Structures Inspection Manual (SIM) published by the Department of Transport and Main Roads (DTMR). Additionally, a custom developed Condition Assessment Plan takes into consideration specific asset portfolio requirements, organisational risk appetite and availability of funding and resourcing.

A rolling program of Level 2 inspections has been implemented since 2010 and is subject to regular review and updates. Inspection frequencies are adjusted to take into account changes in the latest condition rating resulting from either recent inspection or completion of a rehabilitation project. Decommissioned assets are removed from the program and new assets are added including “contributed assets” which are handed over by private developers or other government agencies.

Due to a large number of structures requiring assessment, condition inspections and provision of associated reporting is outsourced to external consultants. The service is usually procured via an open public tender and the contract awarded for three years with a possibility of extension. Robust and detailed technical specifications are paramount to the successful outcome. It is particularly important to define any departures from the standard SIM procedures, clarify exclusions and add any additional business specific requirements. A few examples of such scope clarifications include underwater pile inspection, scour survey, timber drilling survey or ultrasonic testing. Non-standard components may also be added and could include street lighting poles or public utility supports attached to the superstructure.

Another cost effective approach to consider during scoping of the Level 2 contract is identification of all structures requiring special access arrangements such as very high bridges, rail or highway crossings where either specialist equipment must be used or permits obtained to complete the field inspection.

The drawback of using external consultants is lack of continuity in resourcing (inspectors) which may occasionally lead to inconsistent results. Although the SIM procedure provides robust framework for assigning condition states, some aspects are left to the interpretation and subjective judgement of individual inspectors. This highlights an importance of the thorough review process before accepting the Level 2 reports. Specific attention should be paid to structures which changed their overall condition score, and in particular bridges which had no corrective maintenance work done but improved their CS rating.

Considering this, the City has found predictive modelling based on the past condition data, offered by some bridge management applications, is not a reliable source of deterioration forecasting for the purpose of remaining useful life assessment. Level 3 investigations, which include appropriate concrete testing and deterioration modelling, have been found to deliver much more accurate results.

Preliminary review of the Level 2 inspection data allows classification of the identified works into two (2) main categories depending on their complexity and urgency:

- Corrective maintenance for inclusion in an annual Operational Works Program.
- Rehabilitation works as part of the Capital Renewal Program.

### 2.3 Program Development

The City has a dedicated capital Bridge Renewal Program to repair (rehabilitate) or renew (reconstruct) deteriorated structures over a ten year period. The first four years focus on the projects from the highest priority group with preliminary scoping and costing based on the Level 2 condition data. Contingency allowance is set at approximately 15-30% depending on the anticipated size and complexity of the project at the time. Future years, especially year five to year ten, offer less technical detail and budgets are based on the long term financial forecast. As more information becomes available during project planning and upon completion of detailed engineering assessments (Level 3), the scope of works and associated budgets are revised and updated. Subsequently, the Bridge Renewal Program is further refined and, if required, projects are reprioritised and rescheduled in accordance with their latest risk profile.

Presently bridges are considered to be at intervention when the entire structure, a load bearing component or a principal safety component is in condition state CS4 (very poor) or CS3 (poor). Non-compliance with the current engineering standards does not automatically trigger an intervention requirement.

The long term goal, however, is to transition from the corrective (reactive) interventions to preventative (proactive) maintenance. With reference to the “Law of Fives” (as described in SA HB84, 2018) and shown in **Figure 3**, the aim is for the works to be scheduled prior to the corrosion initiation and before the deterioration occurs. This approach offers the best value over the lifecycle of the asset.

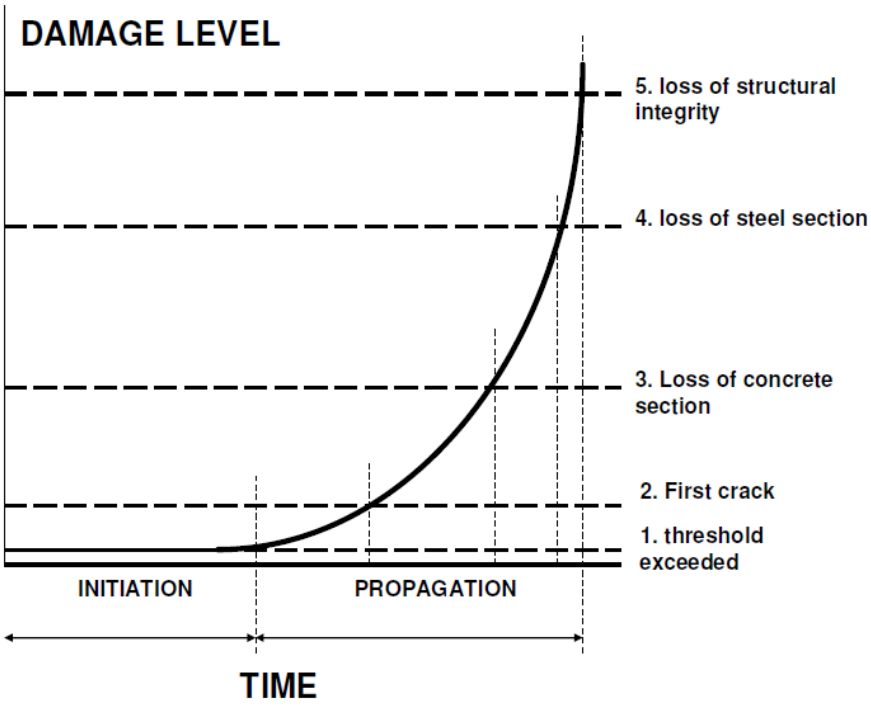


**Figure 3: Typical Stages of Corrosion (Chirgwin, G. *et al,* 2009*)***

In addition to the bridge condition and its risk profile, other factors are taken into account during the program development in order to provide a holistic approach to prioritising identified projects. These factors include the optimum timing of intervention, asset performance, service impacts, safety, legislative requirements, project dependency (in relation to other projects) and deliverability.

Asset profiling is a process of classifying assets into categories (high, medium or low) based on predetermined criteria. Profiling enables a consistent and documented approach to scoring and prioritising bridges for maintenance and renewal activities. It is especially useful if all identified structures have the same condition state rating. The following is a summary of parameters used in the profiling methodology applied to City’s bridges:

- Asset classification (road bridge prioritised over major culvert and pedestrian bridge).
- Road hierarchy (evacuation route, arterial, major collector, minor collector, local).
- Traffic volume (vehicles per day).
- Redundancy (availability of an alternative route).
- Environmental considerations (exposure classification, acid sulphate soil, tidal zone).
- Construction material (principal if combination).

**Table 2** provides details on how each score relates to the individual profiling parameter. The overall profiling score of each bridge is calculated by adding individual scores: C + H + T + A + E1 + E2 + E3 + P. The assets are then categorised into the three risk groups as shown in **Table 3**.

**Table 2: Bridge and Major Culvert Profiling Parameters**

| **Profiling Parameters** | **Ref** | **Score** | | | | |
|---|---|---|---|---|---|---|
| | | **5** | **4** | **3** | **2** | **1** |
| Asset Classification | C | Bridge | | Major Culvert | | |
| Road Hierarchy | H | In evacuation routes | In any arterial roads except for ones in Priority 5 | In any major collector roads except for ones in Priority 5 | In minor collector roads except for ones in Priority 5 | In local urban roads or in rural roads except for ones in Priority 5 |
| Traffic volume | T | ≥10,000 | ≥5,000 and <10,000 | ≥1,000 and <5,000 | ≥100 and <1,000 | <100 |
| Alternative Route | A | Only access for local residents by any vehicle in any situation | Only access to local residence for fire engine and ambulance, but two accesses for normal vehicles in all situations | Two accesses to local residence for fire engine and ambulance but only one access in major storm event | Two accesses to local residence for fire engine and ambulance in any situation | More than two accesses for fire engine and ambulance in any situation |
| Environmental / Exposure | | | | | | |
| ≤ 1km from coast (Y/N) | E1 | Yes | | | | No |
| Acid Sulphate Soil Risk (Y/N) | E2 | Yes | | | | No |
| Within Tidal Water Zone (Y/N) | E3 | Yes | | | | No |
| Material Type (Construction) | P | Steel | Fibre Reinforced Polymer (FRP) | Concrete | Timber | Other |

**Table 3: Asset Profiling Categories for Bridges and Major Culverts**

| Score Range | Profile Category | Profile Category Name | Description/Function |
|---|---|---|---|
| 30-40 | A | High Criticality | These bridges and major culverts are the most critical to the transport network and will be given priority for any first response works required during a weather event, and priority in terms of renewal funding and upgrade considerations. |
| 14-29 | B | Medium Criticality | These bridges and major culverts are less critical than Group A, but will be given priority for any first response works required during a weather event, and priority in terms of renewal funding and upgrade consideration, over Group C. |
| 8-13 | C | Low Criticality | These bridges and major culverts are the least critical to the transport network and will be considered for renewal funding and upgrade consideration after Group A and B. |

To further refine the high risk structures, the following additional factors for road bridges are considered by City engineers during provision of the Bridge Renewal Program:

- Number of defective components within a component group (for example the number of piles per each pier).
- Heavy vehicle crossing managed under the NHVR Scheme.
- Bridge age and corresponding design standards.
- Load carrying capacity / posted load limit.
- Compliance with current engineering and safety standards.

Structures in poor condition (CS4) identified for rehabilitation works but scheduled in future years based on their overall risk profile, are placed on the register and individually managed under bridge specific Structure Management Plans (SMP).

Bridge rehabilitation projects need a highly specialised workforce throughout the life of the project to ensure quality outcomes. Experienced and skilled personnel is required starting from planning and scoping, through to investigations and engineering design, and finishing on the construction execution. The project teams are usually comprised of a combination of internal and external resources. The time necessary to procure specialist external resources affects the overall schedule and needs to be planned for during the program development. In practical terms, availability of resources and funding are the primary constraints affecting the program deliverability. A well thought through Renewal Program ensures assets are maintained in good condition and achieve their expected useful life with the right balance of performance, cost and risks.

## 3 SPECIAL (ENGINEERING) INSPECTIONS – LEVEL 3

### 3.1 Purpose

A Level 3 inspection aims to gain a detailed understanding of the bridge, above what can be provided from a visual inspection alone during Level 1 and Level 2 assessments. Level 3 inspections are not routine (i.e. undertaken by exception) and are only triggered for a specific purpose which is generally unique to the structure.

The objective of a Level 3 inspection is to gain a full understanding of one or more of the following:

- Comprehensive condition assessment and defect identification including components omitted in Level 2 due to access constraints (e.g. underwater, confined space, etc.).
- Deterioration mechanisms and suitable remediation strategies.
- Deterioration modelling (prediction) and remaining useful life.
- Estimation of expected maintenance and capital repairs works.

- Load capacity assessment to verify and understand the structural behaviour and carrying capacity of the bridge.
- Other bridge specific issues, for example confirmation of structural integrity following damage caused by severe weather events.

These investigations will typically include a combination of site investigations and desktop studies; however, the scope can vary significantly dependent on the specific purpose and desired outcomes.

The DTMR SIM recommends these investigations are to be carried out by a “structural engineer”. However, later the manual states inspections must be undertaken under the supervision of an RPEQ “bridge engineer”. Following this requirement needs to be carefully considered and is dependent on the desired outcome and scope of the investigation. For example, determination of remaining life may not be within the skillset of a structural engineer, and a more appropriate choice would be engaging a materials engineer with detailed knowledge of concrete technology and durability. In some cases, the ideal team could be a combination of structural, material, and civil engineers, who each bring their specialist knowledge and skillset.

The asset owner should carefully consider the personnel undertaking the Level 3 inspection, and verify their skills and experience are relevant to the scope of the project.

### 3.2 Planning and Scoping

Typically, an asset owner will request a consultant to undertake a Level 3 inspection of their bridge. Broadly speaking, the scope of the inspection could include (not an exhaustive list):

- Site inspection (i.e. detailed visual inspection).
- Field testing (using non-destructive or destructive means).
- Sampling for laboratory testing (i.e. concrete cores).
- Diver inspections of underwater components including cleaning and removal of marine growth.
- Desktop assessment of the current and historic information relevant to the bridge.
- Load capacity assessment of specific elements or the entire bridge.
- Other bespoke inspection activities specific to the bridge.

These inspection activities may be specified by the asset owner or recommended by the consultant if current issues with the bridge and the desired outcomes are clearly communicated. The commercial implication of Level 3 inspections should be considered, as an over-specified inspection can be a very inefficient use of available funding. Conversely an insufficient or under-scoped Level 3 may result in the asset owner receiving information which is inadequate and does not meet their requirements.

### 3.3 Inspection, Testing and Assessment

Undertaking non-destructive and destructive testing is a necessary supplement to visual inspections, particularly when the objective is to predict remaining life and remediation requirements. It is useful to understand the diagnostic testing techniques which are available, but more importantly, their application and limitations need to be considered.

The recently (2017) published AS5100.8 provides brief descriptions on a selection of destructive and non-destructive test methods which may be used to complement visual inspections. State government publications (e.g. Department of Transport and Main Roads, Roads and Maritime Services, VicRoads, etc.) provide specific additional inspection and testing requirements in some circumstances.

It must be emphasised such testing is not always essential, nor applicable to all scenarios. The type, extent, and frequency of testing must be carefully considered by the asset owner and consulting engineer.

The following sections provide a brief overview of practical experiences with the most common on-site inspection and testing techniques applicable to concrete bridge investigations. The testing discussed is not an exhaustive list.

### 3.3.1 Visual Inspection Techniques

For the purpose of a Level 3 investigation, touching distance visual inspection is preferred. During these inspections, the primary objective is to accurately quantify and measure defects, and assess the severity of observed defects.

There are likely to be instances where the costs associated with gaining safe touching distance access to specific bridge components are not economically or technically justified. In these instances, a variety of remote inspection techniques could be considered as a “first pass” assessment, such as:

- High zoom cameras/binoculars – can be very effective, will generally require a tripod at high zooms to ensure clarity of images.
- Remotely piloted aircraft (i.e. drones as shown in **Figure 4**) – this is a rapidly emerging market with an abundance of technologies available. The primary considerations are safety of the public during drone use (generally by abiding by Civil Aviation Safety Authority’s requirements) and having a highly skilled operator with experience in bridge inspection.
- Borescopes/videoscopes – very useful for cavities or other tight areas (e.g. bridge bearings).
- “Camera on a stick” – a rudimentary technique; however, it can be very cost effective and provide good outcomes. Most useful when live viewing (e.g. on a tablet computer connected via WiFi).

It is important to highlight the limitations of remote inspection techniques, such as their inability to accurately quantify the size of defects requiring repair. In some instances, this project risk can be managed accordingly when procuring the repair works.


**Figure 4: RPA (drone) Bridge Inspection**

### 3.3.2 Underwater Inspection

An often omitted element of a holistic bridge assessment is the underwater component inspection, which typically includes bridge piles and pile caps. In a general sense, permanently submerged concrete is subject to a benign exposure environment, given the availability of moisture to drive corrosion. With this in mind, it may be tempting to exclude underwater inspection to reduce the inspection costs.

Alkali aggregate reaction (AAR) / delayed ettringite formation (DEF), as described in Section 4.1, are internal expansion mechanisms which can lead to concrete cracking, resulting in a direct pathway for moisture and other contaminates to the reinforcement. The only way to identify this cracking requires

direct visual inspection of the concrete surface. To enable inspection, thorough removal of marine growth and cleaning (typically via water blasting and hand tools) is necessary. It may seem logical or economical to clean only a representative sample; however, specific deterioration mechanisms (for example DEF) have been observed to affect bridge piles seemingly at random.

Submersible remotely operated vehicles (ROV) can be a suitable cost effective and safer alternative to human divers, provided marine growth is removed (or isn't present), and where water clarity permits. Generally speaking, ROV's are a useful first pass inspection. However, if quantification of defects such as cracking (i.e. length, width) are needed, ROV's are less beneficial.

Given the potential whole of life cost implications of AAR/DEF on bridge piles, full cleaning and inspection of all submerged bridge elements is prudent to be included in any Level 3 inspection. Where prestressed octagonal piles are used, this is considered essential (as discussed further in Section 5).

### 3.3.3 Delamination (Tap Hammer) Survey

The delamination survey is one of the simplest but most useful inspection techniques. The survey is conducted by striking the surface with a hammer and listening to the sound produced. In most instances it is obvious to distinguish between solid and hollow (drummy) concrete. Delamination can occur most commonly as a result of reinforcement corrosion or construction defects (e.g. blisters in slabs); however, other mechanisms may result in delamination.

AS5100.8 is prescriptive on the type of hammer and technique to be used (striking the surface with the rounded face of a ball pein hammer). The selection of the size and weight of the hammer is not discussed but is of high importance. A small/lighter hammer is better suited for detecting shallow delamination (e.g. low cover, surface defects), whereas a larger/heavier hammer is ideal for deep cover.

Delamination surveys require immediate interpretation by the engineer on site. Due care, skill and experience is necessary to avoid either missing or incorrectly identifying delaminated areas. Common nuances encountered are; changing resonance of the concrete (e.g. if voids are cast into the member, the sound will change in these locations), or if other components cause the sound to reverberate (e.g. steel brackets adjacent to the concrete being struck can mimic the sound of a shallow delamination).

### 3.3.4 Cover Surveys

Cover surveys ultimately aim to determine the layout and depth of reinforcement. The depth of reinforcement is a crucial factor when predicting remaining life. The most commonly used methods for cover surveys are using a cover meter (magnetic), or concrete ground penetrating radar (GPR). Each of these pieces of equipment have their own advantages and limitations, and neither is suitable for all scenarios.

In a basic sense, a cover meter uses magnetism to detect reinforcement. The intensity of the signal received is interpreted by the device to correlate the depth of reinforcement. The most important setting on a cover meter is selecting the correct bar size. A large reinforcement bar at deep cover, will have a similar signal to a smaller bar at shallow depth. The cover meter will interpret the signal based on the selected bar size, and if set incorrectly can result in significant variance between the measured and actual cover. Due to the method of measurement, cover meters commonly suffer interference with highly congested reinforcement, and areas such as laps and intersections. Whilst some cover meters claim to be able to determine bar depth (to plus or minus one bar size), in practice this has rarely been observed to be accurate. Nonetheless the requirement to determine bar size is included in AS5100.8. Good practice will include a localised drill hole or concrete breakout to either verify the bar size, or calibrate the cover meter to the physically measured cover.

GPR's use radio to send a signal into the concrete, and the reflection is received and interpreted by the device (**Figure 5**). The principal of GPR relies on variance in density of the media undergoing scanning; for example, steel reinforcement is approximately three to four times denser than concrete. The primary limiting factor of GPR is the susceptibility to the moisture content of the concrete which

can largely affect the readings. It must be verified the dielectric setting on a GPR correlates to the moisture content of the concrete, which will often necessitate exposing the reinforcement to calibrate the readings. This is of particular difficulty when the moisture content varies within a structural element, such as a bridge pile (tidal, splash and atmospheric).

In an idealised scenario, both devices are used in conjunction to supplement each other. For example, the GPR can be used to initially determine the reinforcement layout, and then the cover meter is used to determine the cover (including a drill hole or breakout if the bar size is not known).

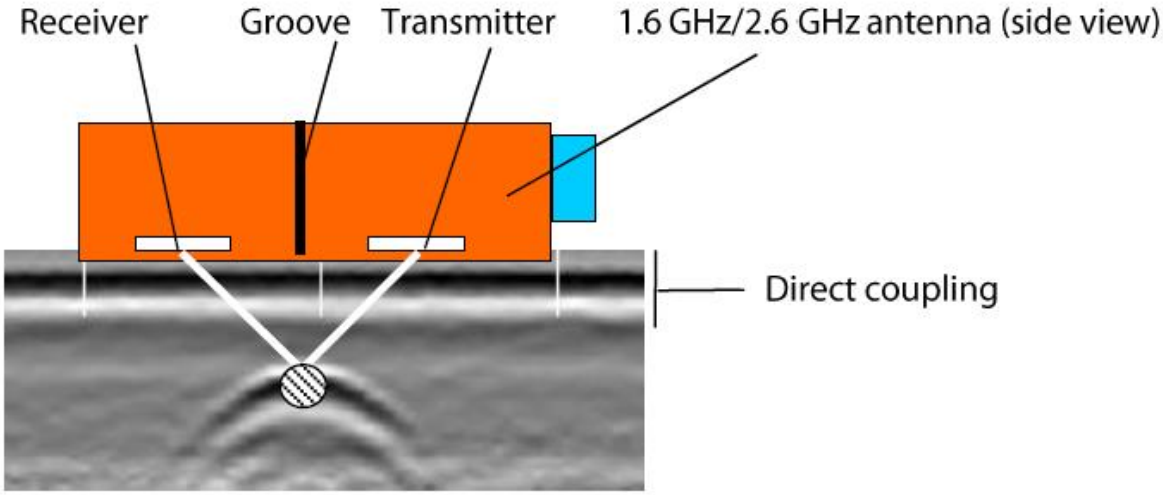


**Figure 5: GPR Locating Reinforcement (source: GSSI Structure Scan Mini II manual)**

A summary of the advantages and limitation of each method are presented in **Table 4**.

**Table 4: Overview of Cover Survey Equipment**

| Equipment | Advantages | Limitations |
|---|---|---|
| Cover meter | - Very accurate for measuring cover<br>- Modern devices are small and light<br>- Relatively inexpensive | - Requires drill hole or breakout to calibrate if bar size is not known<br>- Cannot detect voids<br>- Most devices cannot detect non-magnetic reinforcement (i.e. non-ferritic stainless steel, glass reinforced polymer reinforcement)<br>- Cannot be used when steel fibres or iron containing aggregate are present |
| GPR | - Easy to visually see reinforcement layout on output screen<br>- Can detect voids (generally these need to be >10mm in size depending on the device resolution)<br>- Can detect non-magnetic reinforcement | - Requires drill hole or breakout to calibrate the dielectric setting<br>- Cover reading can have localised inaccuracies based on variable moisture content (not suitable for tidal environment)<br>- Some devices can be very cumbersome (though more compact options are available)<br>- Relatively expensive |

### 3.3.5 Concrete Breakouts

Concrete breakouts are a highly beneficial addition to cover surveys, particularly when the reinforcement arrangement and size is not known. A breakout can range from a small drill hole (typically 16mm diameter) over the reinforcement location (as identified via the cover survey), or can be a larger opening to fully expose the reinforcement bar. Breakouts are a destructive test which may be unappealing to asset owners, however the following benefits are highlighted:

- Provides confidence in the accuracy of the non-destructive cover surveys.

- Enables direct inspection of the reinforcement condition (i.e. if corrosion is present).
- Allows direct measurement of the reinforcement bar diameter (including assessment of section loss if corrosion is occurring).
- Provides a location to undertake neutralisation (carbonation) depth testing.
- The same breakout can be utilised for electrochemical tests (e.g. steel-to-concrete half cell potential surveys).

Small breakouts are typically undertaken using a portable battery-operated hammer drill/jack hammer, ideally combined with a portable angle grinder to saw cut the periphery of the breakout. For larger breakouts, a concrete coring drill can be used over the reinforcement; however, the drill operator must be extremely careful and attentive when drilling to ensure they stop as soon as the reinforcement is reached (steel filings become visible in the water used for cooling, and the water becomes clearer).

Regardless of the method used for breakouts, the area must be immediately reinstated using a suitable proprietary bagged repair mortar.

### 3.3.6 Neutralisation (Carbonation) Depth

Neutralisation depth testing (commonly referred to as carbonation testing) is a quick and simple test conducted on site (can be conducted in laboratory on core samples). Freshly fractured concrete (i.e. core sample or breakout) is sprayed with a solution of phenolphthalein, ethanol and water (**Figure 6**). The phenolphthalein is a pH indicator which will turn magenta in contact with alkaline concrete (where the pH is commonly around 12). If the pH has dropped to below approximately 9 due to carbonation or other mechanisms, the solution will remain colourless.

It should be noted only the cement will change colour and the aggregate will always remain colourless. When measuring the naturalisation depth, it is important to ensure the aggregate is not included.

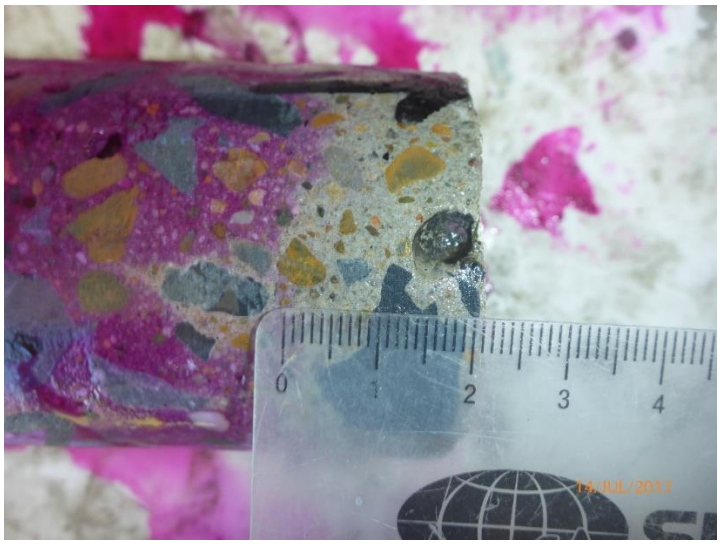


**Figure 6: Neutralisation (Carbonation) Depth Testing**

### 3.3.7 Electrochemical Testing

The Concrete Society Technical Report No. 60 provides excellent guidance on undertaking electrochemical testing on reinforced concrete structures. AS5100.8 briefly describes these tests, however has no detail on limitations, interpretation or techniques to be adopted.

An important aspect covered by The Concrete Society is training of the individuals conducting the tests. Training ensures the engineer is fully aware of the testing fundamentals, correct equipment use, likely sources of error, understanding of factors which can influence results, what and how to record data, and interpretation of the results.

These tests can be time consuming to complete on site and often require stable access for an extended period, and the equipment required can be expensive. For this reason, such testing should only be specified for bridges in environments where the results will be of value, for example a marine structure which may require cathodic protection.

Electrochemical tests will not be discussed in further detail. However, the key consideration is ensuring the personnel conducting the tests have sufficient knowledge and experience, as incorrect testing or results interpretation can negatively or incorrectly influence the rehabilitation strategy for the bridge.

### 3.3.8 Concrete Core Sampling

Retrieval of concrete samples enables laboratory testing to be undertaken. The size (i.e. diameter and length) of the sample is crucial to ensure the tests are completed in accordance with the relevant standards, and the results can be relied upon.

AS5100.8 stipulates core samples shall be 75-100mm in diameter, with the length not specified. This provides little guidance and is likely to result in the incorrect core size being retrieved. A summary of core sample size requirements is provided in **Table 5**. A shorter core with a smaller diameter is preferred from a practical standpoint, as this can be retrieved faster and has less impact on the structure.

**Table 5: Summary of Core Sample Size Requirements**

| Test | Diameter | Length | Notes |
|---|---|---|---|
| Chemical testing (e.g. chlorides, sulphates, cement content) | Minimum 2 times the maximum aggregate size (typically 50mm is suitable) | 10mm past the reinforcement depth | - At least one core should extent 100mm+ to enable measurement of background levels<br>- One core is typically sliced into at least three (3) 10-15mm increments |
| Compressive strength testing | 100mm (75mm is possible but not preferred) | 2 times the diameter (absolute minimum is 1.1 time the diameter however the results require correction) | Good practice will take three (3) samples from a particular member type (e.g. headstock) |
| Petrographic analysis | 100mm (down to 40mm is possible but not preferred as error margin in results increases) | 2 times the diameter (shorter lengths are possible but increases error margin) | Should be taken at locations where evidence of AAR/DEF are occurring |

It is highlighted the size of the hole (i.e. drill barrel size) will be about 5-10mm larger than the core sample. In addition, the length of the core sample will commonly be around 10-20mm shorter than the drilled length (as the core will not fracture at the base).

Other less common laboratory testing techniques are available. Discussions with the laboratory personnel prior to attending site is essential to ensure the correct sample size is retrieved.

### 3.3.9 Load Rating Assessment

A load rating (capacity) assessment (as per AS5100.7) provides an important input to the bridge management strategy and can be a deciding factor on the preferred life cycle strategy for deteriorating bridges. Some level of pragmatism should be adopted, for example an “insufficient” load rating for heavy vehicles may be acceptable to the road authority, if the actual vehicle usage does not represent the theoretical “failed” loading scenarios, or if there are no visible signs of distress in the

under-capacity elements. Asset owners can implement a variety of strategies to manage any identified risks associated with a bridge component. Strategies can include the following:

- Implementation of a Structures Management Plan (SMP) which can be used to nominate an increased inspection regime to monitor components of interest.
- Posting of load limitations and restrictions to heavy vehicles.

# 4 REHABILITATION STRATEGY

## 4.1 Deterioration Mechanisms

The most common deterioration mechanisms experienced by the City's concrete bridges are discussed in the following sections.

### 4.1.1 Chloride Induced Corrosion

Exposure to chlorides (i.e. salt) is an inevitable and unavoidable circumstance for bridges in a marine environment. Over time, these chlorides diffuse through the concrete until eventually they reach the reinforcement. The rate of diffusion ultimately depends on two factors:

- Chloride diffusion coefficient: determined by numerous factors such as mix design, compaction, curing, and others.
- Surface chlorides: this is related to the exposure environment and how much the chlorides can accumulate.

The time at which the chlorides initiate corrosion of the reinforcement will be dependent on the diffusion rate and:

- The depth of cover to reinforcement.
- The type of reinforcement (i.e. prestressed, conventional, galvanised, stainless, etc.).
- The total cementitious content.

Most industry accept chloride models are based on Fick's second law of diffusion. This provides a simple and quick estimate of time to reinforcement corrosion initiation. These models, however, have their own assumptions and limitations, and the validly of the outputs is regularly brought in question. Experience has shown corrosion can occur well below the industry accepted chloride content thresholds, and conversely heavily chloride contaminated bridge piles have been observed to show no signs of corrosion. It is not intended to describe the intricacies of chloride induced corrosion and predictive modelling, as this is not within the scope of this paper. The primary consideration is the information gained from chloride content testing and predictive modelling should be interpreted with caution and considered within the context of visual observations (and other factors). Specialist materials engineers advice should be sought when interpreting or undertaking predictive modelling.

### 4.1.2 Internal Expansion (AAR/DEF)

Internal expansion of the concrete can be caused by either alkali aggregate reaction (AAR) or delayed ettringite formation (DEF). This internal expansion alone may not be problematic; however, the cracking can provide a direct pathway for external contaminants (e.g. chlorides) to the reinforcement. The development of AAR/DEF induced expansion and cracking can take several years or even decades after construction to present. The tell-tale visible sign is 'map' cracking for reinforced concrete elements. For prestressed elements, the cracks will run parallel to the direction of the prestressing, which can easily be confused with reinforcement corrosion induced cracking.

AAR is a result of the aggregates reacting with alkalis within the concrete (or from an external source). Modern concrete mix design can avoid this from occurring by carefully selecting and testing aggregates, and employing other control measures as detailed in HB79 "Alkali Aggregate Reaction—Guidelines on Minimising the Risk of Damage to Concrete Structures in Australia".

DEF occurs when excessive temperatures are experienced during curing (typically greater than 75-85 degrees Celsius dependent on the mix design). The most common situations when this can occur are very large concrete elements (>1m minimum dimension), very high cementitious contents in the mix design (>450kg/m$^3$), or excessive steam curing (i.e. during pre-casting).

Both AAR and DEF are diagnosed using petrographic analysis. Concrete petrography is a highly specialist field and should not be mistaken for geological petrography. The Concrete Society Technical Report No. 71 provides a high level of detail on sample retrieval, conducting the petrographic analysis, and interpreting the results. This test is highly dependent on the skill and experience of the petrographer. It is highlighted concrete petrography is suitable to diagnose the occurrence of AAR/DEF, and will not provide a prognosis on the future risk of AAR/DEF expansion (other test methods can be used for this purpose).

#### 4.1.3 Poor Construction Practices and Design Oversights

Current bridge standards incorporate some learnings of the past and have increased emphasis on design and construction of a bridge to achieve a 100 year service life. Previous design codes were without provisions for long term durability of concrete, particular when exposed to aggressive environments. The large proportion of the City's bridge assets are developer contributed, and designs are typically 'lean' in order to reduce developer costs. It is suspected these designs may not have been subjected to independent design reviews, nor independent supervision during construction. Modern bridges which designed to incorporate overall robustness within the substructure components are less susceptible to major damage associated with exposure to aggressive environments.

The problems observed with the deterioration of concrete elements may be attributed to a combination of old design standards incorporating lower covers, a lack of understanding around AAR/DEF for concrete mix design and curing, poor construction techniques and a lack of quality control around concrete placement, compaction and mix designs. For these reasons, major rehabilitation of bridge stock at typically around 25 to 50 years old is required.

The City's rehabilitation projects offer a unique insight into the 'mistakes' of the past and some of these include:

- Poor quality control during construction resulting in lower strength (and consequently reduced durability) concrete than what is nominated on design drawings.
- Insufficient, excessive or large variations in cover attributed to design standards and quality control issues.
- Incorrect placing/spacing of reinforcement and missing reinforcement attributed to quality control issues.
- Addition of chlorides in concrete mixes (e.g. by using salt water in concrete mix, adding calcium chloride set accelerators, using un-washed marine aggregate, etc.)
- Settlement of relieving slabs due to poor quality control and perhaps design related issues.

### 4.2 Repair Principals

It is of great importance to correctly identify the deterioration mechanism/s affecting the bridge in order to specify suitable repair techniques.

Standards Australia HB84 (2018) "Guide to concrete repair and protection" simplifies the 11 repair principals described in EN1504, into 5 categories:

- Protective coatings or moisture barrier systems.
- Patch repair systems.
- Coating of steel reinforcement.
- Cathodic protection.
- Corrosion control.

Each category is described in detail, with reference to EN1504 made throughout. The latest revision of SA HB84 contains a multitude of improvements of the previous 2006 edition. However, it should be

acknowledged this publication is not a standard and cannot be used as such. The primary limitation is SA HB84 is based off EN1504, which is now a 15 year old European standard (published in 2004), and numerous technologies and product updates have occurred during this period.

It is the role of the design engineer and asset owner to assess the applicability of the guidance information available, and its suitability for application to the bridge under investigation. Ideally this activity is undertaken in close collaboration with material suppliers and contractors, to ensure the most applicable repair type, material, and application method is specified. Nevertheless, some degree of flexibility is often needed during construction to accommodate latent issues and supplier/contractor preferences.

### 4.3 Repair Techniques

Once the deterioration mechanism/s has been confirmed, and the repair principal is selected, the next step is the design of repairs. AS5100.8 (Clause 1.5) describes what should be considered.

*"The design life of repairs, rehabilitation and strengthening shall be based on a lifecycle approach, and shall take into account repeat applications over the lifecycle, future maintenance and repeat access requirements and take into account the residual life of the existing structure."*

AS5100.8 further list factors to consider when undertaking the repair design. **Table 6** briefly describes the most common repair techniques adopted, and the practical experiences and suitable scenarios to use such repairs.

**Table 6: Repair Techniques Overview**

| Repair Type | When is it suitable? | What should be considered? |
|---|---|---|
| Concrete removal and patching | - Repairing of construction related defects<br>- Carbonation induced corrosion damage<br>- Short term holding works in marine environments | - Reinforcement steel lapping may be required<br>- The addition of a coating is recommended for carbonation induced damage (or for aesthetics)<br>- Galvanic anodes for corrosion control may improve the life of patch repairs and prevent incipient anodes in marine environments |
| Crack injection | - For crack widths in excess of 0.2mm in width (cracks up to 0.3mm may be accepted in benign exposure environments)<br>- When there is no delamination, corrosion staining, or spalling associated with the crack | - Is the crack likely to be active or static (i.e. should rigid or flexible injection material be used)<br>- The method of injection (surface packers, "stitching the crack") needs to be considered |
| Concrete coatings | - Controlling moisture and corrosion rates in carbonated concrete<br>- Restricting chloride ingress in marine environments, only if chlorides have not yet started reinforcement corrosion<br>- Bridging cracks <0.2mm in width (ensuring a stripe coat is applied)<br>- To reduce the extent of future concrete patching extent | - Coatings have their own design life and will degrade over time (typically 10-20 years)<br>- There is limited Australian Standards around surface preparation and application, thus reliance on international standards is required (e.g. NACE International) |
| Electrochemical protection | - Heavily chloride contaminated concrete<br>- Long design life (>25 years) required<br>- Most cost effective in larger surface areas (e.g. blade walls, headstocks) | - Choice of protection (e.g. ICCP, HCP, GACP) requires careful consideration and selection<br>- Requires frequent monitoring and maintenance by trained personnel<br>- Typically high upfront capital cost |

| Repair Type | When is it suitable? | What should be considered? |
|---|---|---|
| Pile jacketing (non-structural) | - Marine environment where chlorides have not yet initiated corrosion, and internal expansion (i.e. AAR/DEF) is not occurring<br>- Long design life (>25 years) required | - Not suitable if reinforcement corrosion has commenced unless combined with electrochemical protection measures<br>- Consideration to pour heights and grouting pressures is crucial when selecting the jacket<br>- Typically composite jackets will degrade under UV and require coating<br>- Asset owners are unable to assess the condition of the pile beneath |
| Structural pile encasement | - Reinstate lost capacity due to degradation<br>- Splice damaged section of pile (making original pile redundant)<br>- Confine internal expansion from AAR/DEF<br>- Increase capacity if found inadequate from load capacity assessment | - Construction challenges in erecting formwork and placing concrete<br>- Consideration of the advantages and disadvantages of using temporary or permanent formwork<br>- Designed to AS5100 to achieve a 100 year life |
| Structural changes | - Strengthening of under capacity members<br>- Replacement of severely deteriorated members, in some instances leaving the existing in place<br>- Widening of existing bridges | - Strengthening material may requiring on-going maintenance (e.g. coating of FRP) |
| Replacement | - Lowest whole of life cost when considering repair and strengthening requirements of all bridge components<br>- Other external factors require the bridge to be upgraded (e.g. higher load rating, additional lanes, etc.) | - Site specific constraints need careful consideration and extensive investigation during planning and feasibility stage<br>- High capital cost but reduced maintenance costs |

### 4.4 Strategy Selection

Beginning with the Level 3 engineering assessment report, which provides detailed information on the life cycle costings for individual components, a repair type is selected for each component. The report assigns three options including short (5-10 years), medium (10-25 years) or long term (25+ years) repairs specified for a particular component. The preferred option is the longest term repair offering the best value for money. Lifecycle costs are calculated over a 50 year period. Efficiencies are gained by maximising time between the scheduled rehabilitation works. It is often the case the medium or long term solution is selected. Short term treatments are expensive, do little to extend the service life, and are labour and resource intensive. For example, driven octagonal prestressed piles with significant cracking would normally entail a structural encasement designed with minimal maintenance requirements, extending its service life for 50 years.

However, rarely is only one bridge component repaired, other components deteriorate progressively in marine environments including headstocks, steel barriers, expansion joints and kerbs. For efficiency a bridge capital project usually incorporates all components at intervention.

Once the repair treatments for individual components are nominated, the project cost can be calculated with a higher degree of confidence. The total rehabilitation project cost becomes the subject of a holistic whole of life cost assessment. Comparison is then made with a full replacement cost, assuming a new structure will be designed and constructed to current standards with a minimum service life of 100 years (with only routine inspection and maintenance).

For example, Bridge “X” requires a major maintenance intervention as a result of a Level 3 Engineering Assessment. Structural encasements to all piles are necessary as a long term solution to extend their remaining service life by 50 years. Other components at intervention include headstock repairs, traffic barriers replacement and expansion joint replacement. The estimated capital cost of

rehabilitation works is $1.9 million. **Table 7** below demonstrates how a rehabilitation project is compared with a full bridge replacement project over 100 years. The total lifecycle management costs for a new (replacement) bridge offers better value for money in the long term when compared to a rehabilitation project.

**Table 7: Bridge "X" Whole of Life Cost Comparison***

| **Bridge "X"**<br><br>**Constructed in 1979 (40 years old)**<br><br>**Asset Management Planning Strategy Options** | **Year 1-5**<br><br>**Capital Project Cost** | **Years 1 - 100**<br><br>**Routine Maintenance** | **Year 25**<br><br>**Capital rehabilitation works** | **Years 1 - 100**<br><br>**Inspection and monitoring, Level 1, 2 and 3 reporting** | **Year 50**<br><br>**Bridge Replacement** | **Total lifecycle cost over 100 years** |
|---|---|---|---|---|---|---|
| Option 1: Capital Work Rehabilitation extending remaining life 50 year. | $1,900,000 | $120,000 | $500,000 Protective coatings and component replacement | $350,000 | $2,500,000 | $5,870,000 |
| Option 2: Bridge replacement with a modern equivalent structure | $2,500,000 100 year design life | $90,000 expansion joints and DWS | $0 | $250,000 | $90,000 expansion joints and DWS | $2,930,000 |

*Costings are provided for demonstration purposes only

In the City's experience, bridge rehabilitation costs have been increasing over recent years and in some cases, a full bridge replacement has demonstrated to be the best value for money over the long term.

The key factors in accurately determining 'whole of life' costs of a structure include:

- Precise project rehabilitation costings which require thorough identification and mapping of defects, components at intervention and a well-defined scope of works.
- Accurate bridge replacement costings including provision for high level planning work in order to determine a new bridge configuration.
- Strong knowledge of costs associated with routine monitoring and corrective maintenance practices.

Other considerations for the full replacement of a structure may include:

- Proposed improvements to the road network service levels.
- Heavy vehicle access requirements due to restrictive load limitations.
- Damage sustained as result of a natural disaster.
- Improvements to flood resilience.

Current design standards now focus on concrete durability, design life and provisions for maintenance activities. There is also an emphasis on quality of design, construction and construction supervision, which should alleviate the maintenance burden in the future.

# 5 CASE STUDIES

## 5.1 Hollywell Bridge

### 5.1.1 Description and Background

The bridge was constructed circa 1980 and consists of three spans with transversely post tensioned prestressed deck units and cast in-situ pier headstocks supported on precast prestressed octagonal concrete piles. The abutment walls and wingwalls are cast in-situ, supported on a pile cap which is founded on two rows of piles (**Figure 7**).

A Level 2 (visual only) condition assessment was undertaken in May 2014. Whilst the bridge was rated in Condition State 2 overall, the City had concerns with the defects noted in the report. These defects included horizontal and vertical cracking on the abutment walls and pier headstocks, and localised minor spalling.

Although there were no defects observed in the pier piles, given they were octagonal prestressed (i.e. steam cured), there were residual concerns of internal expansion as a result of AAR/DEF. This risk was considered higher based on the era of construction.


**Figure 7: Hollywell Bridge**

### 5.1.2 Level 3 Inspection Scoping

The objective of the Level 3 investigation was to ascertain:

- The severity and risk of the observed defects.
- Accurate quantification of delamination, spalling and cracking.
- Remaining life prediction of the key structural elements.
- Options analysis and whole of life cost comparison of remediation techniques to achieve a short, medium and long term life extension.

The investigation targeted the bridge substructure components and in particular the pier piles (including full diver cleaning and inspection), pier headstocks, and abutment walls. As there were no observed defects in the superstructure elements, these were excluded from the scope.

Specific activities completed during the investigation were as follows:

- Detailed visual inspection and crack mapping.
- Full delamination survey on all accessible surfaces.
- Pile cleaning and underwater dive inspection of all eight (8) piles.
- Cover surveys (using a concrete ground penetrating radar and cover meter).
- Concrete breakouts at areas of suspected corrosion activity.
- Steel-to-concrete half-cell potential surveys to assess the likelihood of corrosion activity.
- Concrete resistivity measurements to ascertain likely relative corrosion rates.
- Concrete neutralisation (i.e. carbonation) testing on site using phenolphthalein indicator solution.
- Laboratory testing of retrieved concrete core samples for chloride content and petrographic analysis.

Following the completion of the inspection, a load capacity assessment was added to the scope of the investigation due to further defects uncovered.

### 5.1.3 Engineering Investigation Outcomes

The key findings from the Engineering investigation are summarised in **Table 8**.

Overall, it was found the structure was in much worse condition than indicated by the visual only (Level 2) investigation. Significant variations in condition were as follows:

- Extensive delamination in the pier headstocks and abutments was uncovered, which was identified by the full delamination survey.
- Active reinforcement corrosion resulting in steel section loss was confirmed at concrete breakout locations.
- Severe cracking was uncovered beneath the marine growth on the piles.
- Heavy concrete carbonation and elevated chloride concentrations were measured.

The findings illustrate the limitations of a visual only inspection. Whilst a Level 2 inspection is a cost-effective method of understanding the condition of the bridge asset portfolio, in some cases the condition may be significantly worse (or perhaps less severe) than is visible.

**Table 8: Hollywell Bridge Inspection Findings Summary**

| Bridge Element | Key Findings | Image |
|---|---|---|
| Pier piles | - 50% (i.e. 4 of 8) were found to have severe vertical cracking between mean water surface and riverbed level. Petrographic analysis confirmed the cracking to be a result of internal expansion due to delayed ettringite formation (DEF).<br>- Very high chloride concentrations in the tidal zone were present at the depth of reinforcement, which exceeded the industry accepted corrosion initiation threshold (although corrosion induced damage was not observed) |  |

| Bridge Element | Key Findings | Image |
|---|---|---|
| Pier headstocks | - Extensive corrosion induced delamination and cracking with a total area measured at approximately 12m$^2$<br>- Up to 40mm of concrete carbonation (with 45mm cover)<br>- High chloride concentrations at depth of reinforcement (which are suspected to include a high level of cast in chlorides)<br>- Confirmed reinforcement corrosion at concrete breakout locations<br>- Erroneous steel-to-concrete potential findings due to heavy carbonation |  |
| Abutment walls and wingwalls | - Very similar condition to pier headstocks<br>- Approximately 34m$^2$ or corrosion induced delamination and cracking<br>- Carbonation depth of 42mm exceeding the minimum measured cover of 34mm |  |

Based on the findings, the City requested a desktop structural assessment to better understand the structure. A load capacity assessment was undertaken which revealed:

- Modern loading (T44) is approximately 33% higher than the bridges original design loading (MS18).
- The pier headstocks were showing a theoretical flexural failure under specific load scenarios, which did not correlate with site observations.
- The deck units and pier piles (axial loading only) had sufficient capacity.

The condition assessment and load capacity assessment were used as the basis to formulate appropriate remedial options.

### 5.1.4 Options Analysis and Remediation Approach

Remedial options were analysed for each of the bridge components inspected, as summarised in Table 9. The objective of the remedial works was to extend the remaining service life, at a lowest whole of life cost.

Based on the observed DEF expansion, a structural encasement was essential for at least 4 of the bridge piles. The remaining 4 piles required moisture and chloride exclusion in the tidal and splash zones due to the elevated chloride levels.

**Table 9: Hollywell Bridge Options Analysis**

| Bridge Element | Option 1 (5-10 years) | Option 2 (10-25 years) | Option 3 (25+ years) |
|---|---|---|---|
| Pier piles | - Structural encasement (4 piles)<br>- Coating (4 piles) | - Structural encasement (4 piles)<br>- Durability encasement (4 piles) | - Structural encasement (8 piles) |
| Pier headstocks and abutments | - Concrete repairs and crack injection | - Concrete repairs and crack injection<br>- Protective coating | - Concrete repairs<br>- Electrochemical protection (ICCP) |

The options analysis also considered looking at the bridge holistically. Given the severity and extent of defects, the option for renewal (i.e. bridge replacement) was considered. The whole of life costs of a replacement bridge was compared with the capital costs of undertaking the major repair works, combined with the predicted maintenance and repair works over the life of the structure.

When considering the whole of life costs, site specific constraints, project risks, and advantages/disadvantages, it was determined the most effective long term management strategy was to replace the bridge. Interim measures to manage the bridge (i.e. Structure Management Plan) were agreed to enable the replacement to be planned and scoped accordingly.

### 5.1.5 Project Successes and Lessons Learned

The Level 3 investigation revealed the extent, severity and cause of the known defects was significantly more severe than originally reported under the visual Level 2 Condition Assessment. This project highlighted the importance of delamination surveys and full access to the bridge. Full cleaning of all piles was paramount in determining the best outcome for the bridge, particularly as not all piles were experiencing DEF induced cracking.

Whilst the bridge was not determined to be presently in a structurally deficient condition, remaining life analysis revealed significant remedial works were expected to be necessary to extend the life of the bridge beyond 25 years.

The whole of life costing analysis demonstrated replacement would be the most economical option over a 50 year period (noting a new structure in accordance with AS5100 would be designed with a 100 year life).

Key lessons learnt included:

- Inspections need to allow enough time for the investigation works to fully record and map all defects. The time spent on site was significantly more than estimated due to the vast extent of defects uncovered. Understanding the extent of defects was crucial in understanding the full scope of repairs needed.
- Full cleaning of all piles to remove marine growth. Whilst generally below lowest astronomical tide is considered to be a benign exposure environment, it provides perfect conditions for internal expansion (i.e. DEF/AAR).
- Underwater pile inspection including a clean-off must be included as part of a Level 3 investigation for bridges over salt water. The two “sister bridges” constructed at similar times by the same developer will be the subject of a future Level 3 investigation and assessment to verify the bridges whether they suffer from the same levels of deterioration and schedule within the bridge renewal program as required.

# 6 SOUTHPORT BRIDGE

### 6.1.1 Description and Background

The bridge was constructed circa 1968 and consists of three spans with transversely post tensioned prestressed deck units, and cast in-situ pier and abutment headstocks supported on precast prestressed octagonal concrete piles (**Figure 8**).

A Level 2 (visual only) condition assessment was undertaken in June 2014. The only observed notable defects were cracking in the pier and abutment headstocks, and cracking/spalling of the concrete service supports. The piles were rated in Condition State 1 with no defects observed at the time of inspection.


**Figure 8: Southport Bridge**

### 6.1.2 Level 3 Inspection Scoping

This project was procured by the City as a design and construct (D&C). The works initially included a Level 3 investigation, primarily to quantify defects and provide justification for or against applying a protective coating system to the headstocks, based on the whole of life costs.

Included in the investigation was full cleaning of all 12 pier piles to enable inspection. The piles were (almost) fully exposed during the spring low tide.

Specific activities completed during the investigation were as follows:

- Detailed visual inspection and crack mapping.
- Full delamination survey on all accessible surfaces.
- Pile cleaning and inspection of all twelve (12) piles.
- Cover surveys (using a concrete ground penetrating radar and electromagnetic cover meter).
- Concrete breakouts at areas of suspected corrosion activity.
- Steel-to-concrete half-cell potential surveys to assess the likelihood of corrosion activity.
- Concrete resistivity measurements to ascertain likely corrosion rates.
- Concrete neutralisation (i.e. carbonation) testing on site using phenolphthalein indicator solution.
- Laboratory testing of retrieved concrete core samples for chloride content, sulphate content, cement content, and petrographic analysis.

Following the completion of the initial inspection, severe defects were uncovered in the piles underneath the marine growth. Consequently, a load capacity assessment of the piles was added to the scope of the investigation, and additional pile inspection was undertaken to better understand the full extent of defects.

### 6.1.3 Engineering Investigation Outcomes

The findings from the investigation generally correlated with the Level 2 inspection, with the exception of the pier piles.

Upon cleaning of the piles, significant vertical cracking (>2mm in width) and corrosion product staining was uncovered on 3 of the 12 piles from approximately mean water surface and below. Excavations completed around 4 piles which were buried to a higher level revealed a further 2 piles with vertical cracking. Even with the severe cracking observed, only very localised areas were experiencing delamination, which indicated reinforcement corrosion may not be the primary cause of the cracks. Petrographic analysis of core samples taken from these piles later uncovered DEF induced internal expansion was the primary cause of the cracking, which was consistent with the crack pattern expected given the piles were prestressed. Additional petrographic analysis was undertaken on a visually sound pile, which showed deleterious DEF was not occurring.

The worst areas (i.e. widest cracks and corrosion staining) were investigated further by breaking out concrete and exposing the prestressed reinforcement. This uncovered ‘black rust’ had caused complete or partial section loss in numerous prestressing strands and ligatures. This ‘black rust’ was a result of anerobic corrosion, which is non-expansive and can often go undetected as it does not show the typical signs of reinforcement corrosion (i.e. spalling). Steel-to-concrete potential surveys confirmed corrosion probability was high in other cracked areas, and surface resistivity measurements showed a moderate to high corrosion rate was predicted.

It was concluded the DEF induced cracks had been wide enough to allow a significant amount of chloride ingress in this area, which reduced the concrete resistivity and created a macro-cell (where the top portion of the pile is the cathode, and the corroding area at low water is the anode). This has created ideal conditions for anaerobic corrosion to occur, which resulted in the observed non-expansive ‘black rust’. Over time, the ‘black rust’ made its way through the capillaries (and the cracked concrete) through to the surface (**Figure 9**). Once exposed to oxygen, the corrosion product changed to the more commonly observed “red rust”.

A load capacity assessment was promptly undertaken on the piles, which indicated structural encasement was necessary to resist lateral loads (i.e. from severe flood events).

This project demonstrates how a simple task such as cleaning the piles, can result in uncovering defects which require immediate rectification to ensure the structural integrity of the bridge is maintained.

As a result of the investigation, structural encasements were designed and constructed for the 5 piles experiencing DEF induced expansion and anerobic corrosion.

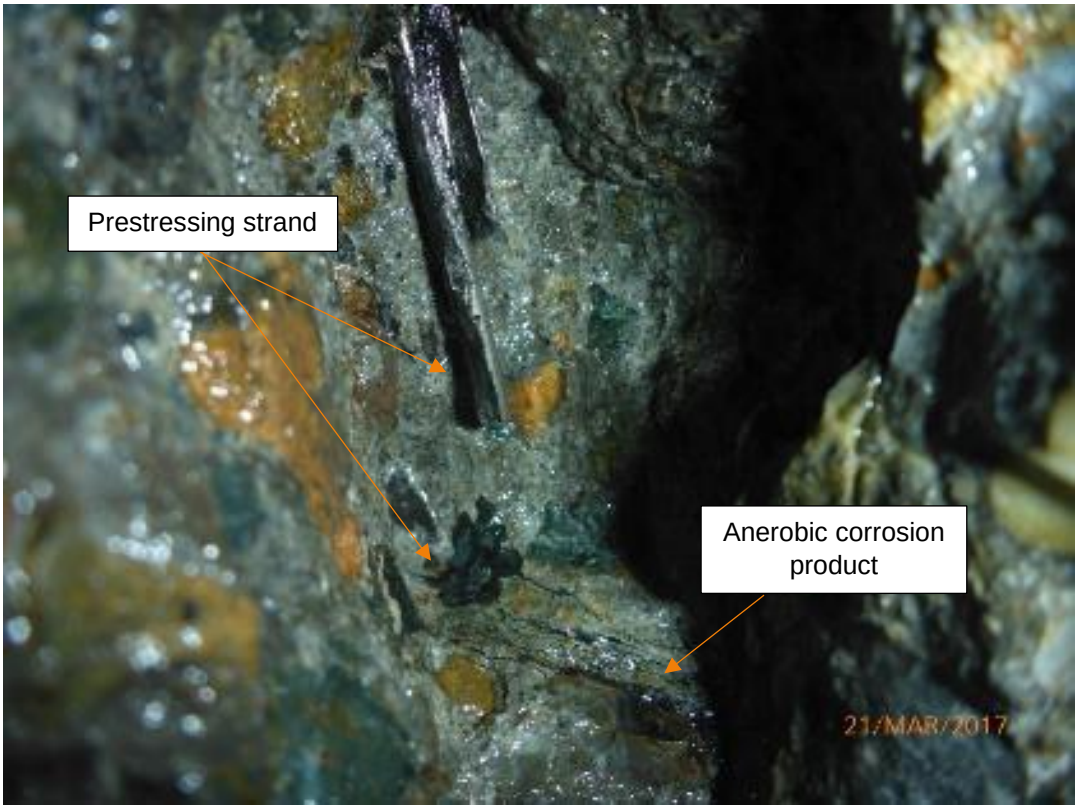


**Figure 9: Anerobic Corrosion to Prestressing Strands**

### 6.1.4 Project Successes and Lessons Learned

The pile cleaning uncovered serious defects which went completely undetected during the Level 2 inspection. This was not due to any fault of the inspectors, but simply because these defects were covered by a thick layer of marine growth.

The petrographic analysis results were a vital piece of information to confirm the cause of the cracking. It is not uncommon for experienced engineers to incorrectly diagnose vertical cracking on bridge piles as being caused by chloride induced corrosion. Typically, DEF/AAR induced internal expansion is associated with “map” cracking on reinforced concrete elements. However, in prestressed concrete (such as bridge piles and deck units), the stresses caused by internal expansion concentrate in such a way to result in cracks forming parallel with the strands (in the case of a bridge pile, vertical cracks). For this project, the petrographic analysis explained why only 5 of 12 piles were cracked, even though they were exposed to the same conditions. It is postulated the steam curing process likely employed for the piles was not undertaken with a high degree of care, which resulted in DEF only occurring on piles which were “over-cooked” (i.e. temperatures exceeded 75°C). The importance of using an experienced petrographer for such analysis is imperative.

Key lessons learnt included:

- The D&C contract was an efficient use of resources which minimised procurement activities and avoided a two staged approach, and in turn saved time and money. The investigation component was thorough, leading to the identification of additional unforeseen defects resulting in five (5) structural pile encasements. The consultant and contractor were highly experienced in concrete repair and rehabilitation work and were able to provide an economical and durable pile encasement design.
- The drawback of not undertaking a full Level 3 inspection and underwater survey in advance was a major budget increase during construction phase, considerably exceeding the project contingency allowance. Additional funding was obtained deferring another (less critical) project within the program.

- Traditional structural encasements were employed using steel temporary forms with galvanised steel reinforcement cages. This allows for inspection and access in the future.
- Working around low tides proved challenging from a programming and construction perspective. Excavation and pile preparation work below the bed level in the tidal zone could only occur for a very short period of time at the low tide only.
- Key to the successful outcome was robust quality control written into the design documentation and enforced by a Superintendent. The Consulting engineer was also able to ensure quality aspects including surface preparation, materials and concrete placement were correctly undertaken.

## 7 BIGGERA WATERS BRIDGE

### 7.1 Description and Background

The bridge was constructed circa 1960 and consists of three spans with transversely post tensioned prestressed deck units, and cast in-situ pier and abutment headstocks supported on cast in-situ piles. The cast in-situ piles utilised precast concrete cylinders (i.e. pipes) as a permanent formwork (**Figure 10**).

**Figure 10: Biggera Waters Bridge**

A Level 2 (visual only) condition assessment was undertaken in 2014 and rated the bridge in CS4, primarily due to severe cracking to the pile cylinders and headstocks. A Level 3 Engineering investigation and assessment was required to identify the failure mechanism and rehabilitation options.

### 7.2 Level 3 Inspection Scoping

The Level 3 investigation, assessment and detailed design was procured by the City as a separate contract to the subsequent construction contract. The primary reasons were twofold; the first being the budgetary constraints and programming meant investigation and design was to taken place in separate financial years to the construction works, and secondly undertaking thorough investigation and design works minimised the construction project variations risk due to unforeseen (latent) issues. Activities completed during the investigation were as follows:

- Detailed visual inspection and crack mapping.
- Full delamination survey on all accessible surfaces.
- Pile cleaning and underwater inspection of all four piles.
- Cover surveys (using a concrete ground penetrating radar and electromagnetic cover meter).
- Steel-to-concrete half-cell potential surveys to assess the likelihood of corrosion activity.
- Concrete resistivity measurements to ascertain likely corrosion rates.
- Concrete breakouts at areas of suspected corrosion activity.
- Concrete neutralisation (i.e. carbonation) testing on site using phenolphthalein indicator solution.
- Laboratory testing of retrieved concrete core samples for chloride content, sulphate content, pH cement content, and petrographic analysis including that of the pile core.

### 7.3 Engineering Investigation Outcomes

#### 7.3.1 Piles

Core samples taken from the parent piles indicate significant expansion cracks have occurred as a result of AAR product formation (**Figure 11**). The petrographic analysis results confirmed the presence of alkali aggregate reaction (AAR) of the core concrete of the piles by way of AAR gel formation, most likely from the coarse aggregate identified as crushed andesite.

It was concluded the observed vertical cracking to the existing cylinders was a combined consequence of two issues. The first being the result of the outward expansion force induced by AAR and the second being the presence of cast in chlorides, believed to be a result of seawater being used in mixing. Without sufficient confining reinforcement in both the parent pile and the cylinder, concrete expansion was inevitable.

A load capacity assessment was undertaken on the piles based on full removal of the damaged cylinders to ensure ongoing structural integrity during installation of new structural encasements around all four piles. The bridge had to remain open to traffic throughout the entire construction period, and the assessment had confirmed no temporary load limit was necessary. However, strict construction sequencing had to be followed.


**Figure 11: Vertical Cracking to Piles**

The new structural pile encasement consisted of a galvanized reinforcement cage designed to withstand AAR bursting pressures, and a high strength concrete with suitable cover due to exposure to an aggressive marine environment. A bespoke self-compacting micro concrete mix was designed which was pumpable and could be placed underwater using a tremie. A proprietary glass fibre reinforced polymer (GFRP) permanent form was specified.

#### 7.3.2 Headstocks

The cracking was predominately around 0.5mm in width but increased to 1.5mm in some areas. It was attributed to a combination of mild AAR, and early age plastic shrinkage due to excessive cover to the reinforcement. There was no evidence of reinforcement corrosion due to chloride ingress.

### 7.4 Options Analysis and Remediation Approach

Short, medium and long-term options for rehabilitation of both the piles and headstocks were developed as part of the engineering assessment report. Lifecycle costings for individual components over 50 years were used to determine the best overall strategy for rehabilitation. An example is provided in **Table 10**.

**Table 10: Headstock treatments life cycle cost and risk (example)**

| **Item** | **Short Term (5-10 years) Crack injection** | **Medium Term (10-25 years) Crack Injection and protective coating** | **Long Term (25-50 years) Cathodic Protection** |
|---|---|---|---|
| Access and preliminaries | $30,000 | $40,000 | $50,000 |
| Capital repair cost | $32,000 | $35,000 | $300,000 |
| Average yearly maintenance cost | nil | nil | $2,000 |
| Ongoing maintenance @ 10 to 50 years | $110,000 | $70,000 | $30,000 |
| Total 50 year cost | $172,000 | $145,000 | $480,000 |
| Risk | High | Low | Medium |

Headstock rehabilitation treatments included selection of the medium term repair consisting of crack injection and a protective coating based on the lifecycle costs above.

Structural encasements were selected as the long term pile rehabilitation option. Whilst not the least expensive over 50 years, it presented the lowest risk option and resulted in little to no ongoing maintenance requirements over its service life. Short and medium term repair options involved higher levels of risk associated with ongoing monitoring and maintenance.

This 60-year-old bridge included other components requiring maintenance intervention. The full scope of capital rehabilitation works comprised of steel and timber walkway refurbishment, application of protective coatings, installation of new expansion joints, new deck wearing surface, and abutment protection repairs. The remaining service life of the structure was extended by an additional 50 years.

### 7.5 Project Successes and Lessons Learned

The correct diagnosis of the cause of deterioration is critical to the successful design of repairs. Diligent scoping, to ensure the correct testing is undertaken without creating unnecessary investigations, was attributed to the involvement of experienced specialist materials and structural engineers.

A capable and highly experienced contractor was engaged to undertake the construction works involving rehabilitation of a variety of the bridge components.

Management of the navigable channel, recreational waterway users, pedestrian movements and vehicular traffic control during the variety of stages required early planning, approvals and good communication with all the project stakeholders.

Removal of the existing concrete cylinders was undertaken by specialist divers, but the time taken was considerably longer than originally estimated.

Selection of a proprietary GFRP permanent formwork jacket provided cost saving benefits; however, it was untried at this size/diameter. Innovative engineering and construction techniques were applied to overcome potential bursting pressures which became apparent during the first jacket pour. On concrete placement of the first pile, the proprietary jointing system restraining the GFRP jacket reached failure point near completion of the pour. Improvements to the placement method and

bolstering of the jacket were required if the project was to continue using the jackets. A contractor initiated improvement was the bespoke steel brace designed and fabricated to reinforce the joint during the next jacket installation. An anti-washout additive was incorporated into the concrete mix to remove any concerns with environmental contamination should the jacket fail.

The remaining three pile encasements were installed successfully as shown in **Figure 12**.

**Figure 12: Completed Structural Pile Encasements**

## 8 CONCLUSION

The following conclusions have been made:

- All pre-1980's bridge structures have been the first to undergo major rehabilitation. There have been lessons learned during all phases of the projects from inception, planning, investigation, design and construction.
- Level 3 inspection findings should be used to develop and refine the Bridge Renewal Program in a sustainable and cost effective way, ensuring major maintenance interventions occur at the most optimal time in the lifecycle of the asset.
- Rapid deterioration of concrete occurs in aggressive salt environments typical of Gold Coast waterways. The most common deterioration mechanisms, investigation and testing methods used have been summarised.
- The importance of understanding the advantages and disadvantages associated with diagnostic inspection and testing of concrete was discussed, with practical considerations and techniques described.
- The City has a robust evaluation process consisting of routine Level 1 and Level 2 condition assessments and by exception, Level 3 engineering investigations scoped to achieve the project objectives.
- The City relies upon specialist materials, structural and civil engineers who specialise in bridge rehabilitation to further refine the scope Level 3 investigations and design durable repairs for defective components.
- Essential forward planning activities and selection of repair options, including the whole of life analysis, will ensure the best overall value for money over the asset lifecycle. Full replacement of a structure can be a feasible solution and the best course of action for an ageing deteriorated bridge asset.
- Implementation of robust quality control methods is imperative to a durable and successful rehabilitation project. Rehabilitation projects have achieved multiple benefits including maintained structural integrity, improved public safety, extension of service life by up to 50 years and enhanced aesthetics

## 9 REFERENCES


- Chirgwin, G. *et al* (2009), A Global Study of Chloride Induced Corrosion Within the RTA's Reinforced Concrete Bridge Stock, Convention Management NZ
- Concrete Society (2004), Technical Report No. 60 Electrochemical tests for reinforcement corrosion
- Concrete Society (2010), Technical Report No. 71 Concrete Petrography
- Department of Transport and Main Roads (2016), Structures Inspection Manual
- Standards Australia (2017), AS5100 Bridge Design (Set)
- Standards Australia (2018), SA HB84 Guide to concrete repair and protection
- Standards Australia (2015), SA HB79 Alkali Aggregate Reaction—Guidelines on Minimising the Risk of Damage to Concrete Structures in Australia


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