Investigating the correlation between pre- and post-demolition assessments for precast, post-tensioned beams in service for 45 years

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The thesis contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text. I give consent to this copy of my thesis, when deposited in the University Library, being made available for loan and photocopying subject to the provisions of the Copyright Act 1968.

Torill Myra Papè
I dedicate this thesis to my loving and supportive husband Anton, who has put on hold his own dreams to allow me to follow mine.

I’m blessed to share this life with you.
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Finally, I give thanks to my God for the opportunity and my inspiration - through Him all things truly are possible.
Roads to Sorell

Welcome, to Van Diemans Land
you rogues that humble Britain, who finally made a stand,
to send you to Port Arthur upon the brink of hell
where on the road to Hobart is this little town Sorell.

Founded in the eighteen hundreds, Sorell cant ignore
the bushranger Matt Brady back in eighteen twenty-four,
when he stormed the settlement wreaking havoc on the law.
The most infamous bushranger, roads to Sorell ever saw.

Now the rogues have gone but their history lives today.
Milled wheat is just a memory for the folk up Sydney way.
Stone buildings dare the southerlies gusting up storm bay,
where we cross Pittwater Shallows on the Sorell Causeway.

The first Sorell bridge was built, back in eighteen seventy-two.
Eight years it took to build and many never seen it through,
for they became the bankrupts, and now that bridge has gone
where above sea stars and oyster farms, Sorell’s moving on.

Two years, two long years have passed.
This new Sorell causeway bridge is opened up at last
and McGees bridge to Hobart joins Sorell’s noted days
of bushranging convicts, and two ruined causeways.

Now the rogues have gone but their history lives today
Milled wheat is just a memory for the folk up Sydney way
Stone buildings dare the southerlies gusting up storm bay
Where we cross Pittwater Shallows on the Sorell Causeway.

Where we cross Pittwater Shallows on the Sorell Causeway.
Between Hobart and Port Arthur is this little town Sorell.

Author Unknown
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Abstract

The Sorell Causeway Bridge, located in Tasmania, Australia, was completed in 1957 and was the first precast, post-tensioned bridge constructed in Australia. However after only 45 years of service, the bridge was replaced due to increasing concerns surrounding the level of corrosion of the prestressing strands in the beams. Prior to its decommission, an extensive and costly investigation program was carried out on the bridge in an attempt to determine the rate of deterioration and establish the remaining margin of safety. Despite the number of investigations and the resulting large quantities of information, the questions surrounding the safety of the bridge remained unanswered. The issue is thus raised: what do field investigations of reinforced or prestressed concrete structures with evidence of corrosion deterioration tell engineers about the actual condition of the structure?

Three beams of varying condition (good, average, poor) were salvaged from the bridge demolition for further detailed examination to investigate the degree of correlation between pre-demolition field investigations and the physical condition of the steel post-demolition. The investigations included the use of conventional non-destructive techniques such as cover, half-cell potential and concrete resistivity surveys, and destructive techniques such as chloride profiling, carbonation depth measurement, and full-scale load testing, all of which were used to determine the likely risk of corrosion and likely corrosion rate for each beam. The results of these investigations were subsequently reviewed in relation to the physical condition of the steel.

In general, all non-destructive tests were found to be inconclusive in relation to evidence of steel corrosion and the corrosion risk guidelines recommended in the literature. It was also apparent that these techniques were incapable of detecting steel pitting, a primary concern for the current investigation. Chloride profiles were variable and inconsistent in relation to steel corrosion and
the chloride thresholds recommended in the literature. Carbonation was found to exist at prestressing levels in some locations and appeared to be influenced by the orientation and geometry of the beams. All beams did not achieve the estimated design capacity and corrosion had significantly impaired the ultimate capacity and ductility of beams in the worst condition.

Aerobic and anaerobic corrosion products were identified via XRD analysis. These included Magnetite, Goethite, Akaganeite, Lepidocrocite, chloride-based Green Rust (I), and Iron (III) Oxide Chloride. The phenomenon of “chloride weeping”, or droplets of highly acidic ferrous chloride, was observed forming on some steel/concrete interfaces on freshly cut concrete surfaces. Several other unexplainable observations were made during the course of the present investigations. These included bright, metallic pit surfaces; pits with concentric rings; black, wet rust covering bright, metallic surfaces; and unusual pitting profiles. A possible explanation for these observations may be the implication of microbiological activity in the corrosion process. Further research is required to confirm these observations.

**Keywords:** prestressed concrete, field investigations, corrosion, non destructive testing, concrete cover, half-cell potential, concrete resistivity, carbonation, chloride profiles, load test, flexural capacity, corrosion products, pitting, ferrous chloride, microbiological activity
Chapter 1

Introduction

The Australian road network comprises over 800,000km of public roads, within which there are some 37,000 bridges [52]. Almost a third of these bridges were constructed between 1948 and 1976 and over 75% are classified as either reinforced or prestressed concrete. These statistics reflect the construction boom after the Second World War and the commercial viability of prestressed concrete as an efficient construction material. The Sorell Causeway Bridge in Tasmania was part of this new era in Australia. The bridge was completed in 1957 and was the first post-tensioned bridge of its kind in Australia [100]. The motivation behind the design was to create a long-lasting and maintenance-free structure, and prestressed concrete encapsulated this sentiment at the time.

However 45 years after the bridge was opened, it was decommissioned and a new structure erected in its place. This was the result of concerns regarding the extent of corrosion of the prestressing strands in the beams and diaphragms, potentially threatening the safety of the bridge. Prior to demolition, a substantial and expensive investigative program was conducted on the bridge. This program was triggered by the discovery of a 2m longitudinal crack in the web of a beam that appeared to follow the trajectory of the post-tensioning tendons. The investigations included half-cell potential, resistivity and covermeter surveys, coring, chloride profiles, structural assessments, and load testing. Additional reports were commissioned and expert opinions canvassed to review previously obtained data in the hope of more accurately defining the condition of the bridge and estimating the remaining life of the structure. Despite the number of investigations, the condition of the bridge still remained unclear. As the bridge was located on a critical
arterial road, combined with the risks posed to public safety and the social and economic impacts if the bridge collapsed, the bridge was deemed unfit for service and the decision was made for its replacement.

Approximately A$5.5 billion was spent in 2003/04 in maintaining and improving the national road network in Australia, of which 2.5% of the cost is attributable to bridge maintenance [52]. In 1997, the Tasmanian State Government was reported to have spent approximately A$9 million per annum in maintaining its bridge network [49]; in 2000, it was estimated that 16% of concrete structures in the state network showed evidence of deterioration due to corrosion, with 4% of structures (or 26% by value) scheduled for short-term replacement due to severe corrosion [110, 248]. In relation to condition assessments, no specific costs have been published in Australia to date, but based on these observations it is suggested that substantial costs are involved to understand and then specify maintenance works. Therefore, it is seen as beneficial to improve understanding and interpretation of field investigations in relation to corrosion to avoid a repeat of the occurrences on the Sorell Causeway Bridge.

The objective of this thesis is to examine the degree of correlation between pre- and post-demolition investigations which have been conducted on precast, post-tensioned beams retrieved from the Sorell Causeway Bridge after its demolition. More specifically, this project will investigate the correlation between the degree of corrosion in the reinforcing and prestressing steel retrieved from the beams and commonly available corrosion inspection methods utilised for predicting the likelihood and severity of corrosion in concrete, which are listed below:

- Non-destructive methods: Visual inspections, crack mapping, and covermeter, half-cell potential, and resistivity surveys
- Destructive methods: Load tests, chloride profiles, and carbonation depths

Corrosion inspection results obtained independent of those determined by the Tasmanian road authority have been the predominant focus of the current investigation, however some comparative findings between the two data sets has also be presented. In addition to the correlation study, findings relating to the physical condition of the steel have been presented.
In relation to corrosion assessments of reinforced and prestressed concrete structures, much of the literature derives recommendations from laboratory-prepared specimens that rely on accelerated corrosion techniques and often do not account for “real-life” in-service loading. The present project thus presents a unique opportunity to study in detail an actual bridge that was located in a marine environment and has been in service for 45 years. It is also, to the author’s knowledge, the first post-Second World War concrete bridge in Australia to be demolished.

The work contained herein is structured as follows: Chapter 2 provides an overview of the Sorell Causeway Bridge, including its construction details and a history of its deterioration. It also includes a review of all previous field investigations. Chapter 3 provides a summary of the current understanding of the corrosion process of reinforcement in concrete. Experimental test results obtained prior to demolishing the beam specimens are given in Chapters 4 and 5; non-destructive test results typical of those determined prior to the bridge being decommissioned are reported in Chapter 4, where load test results and chloride/carbonation profiles are discussed in Chapter 5. Chapter 6 details the findings in relation to the physical condition of the steel retrieved after demolition. All results are discussed in Chapter 7, followed by conclusions and recommendations for further research in Chapter 8.
Chapter 2

Project Background

2.1 Introduction

Prior to discussing results contained herein, it is important to provide some background on the Sorell Causeway Bridge. This chapter will provide information relating to the bridge’s location (Section 2.2), historical development and design information (Sections 2.3 and 2.4), and the series of events leading up to its decommission (Section 2.5). A brief overview of selected pre-demolition investigation data has been provided in Section 2.6. Finally, Section 2.7 reviews case studies similar to the present study. For information pertaining to the nomenclature used within this chapter, see Appendix A.

2.2 Bridge Location & Environment

The Sorell Causeway is located in South-East Tasmania, Australia and is managed by the Department of Infrastructure, Energy & Resources, Tasmania. It spans across the Pitt Water estuary between Midway Point and Pitt Water Bluff, approximately 20 kilometres north-east of Hobart, Tasmania (Figure 2.1). It comprises a crucial link of the Tasman Highway, connecting Hobart with the popular east coast and significant tourism attractions, such as Port Arthur. The Sorell Causeway Bridge itself was located adjacent to Midway Point (on the eastern side of the causeway). The causeway is approximately 1600m long, where the bridge takes up almost a third of that length. It attracts 13500 vehicles per day Average Annual Daily Traffic (AADT), where traffic comprises of 4% semi trailers, 8% heavy rigid vehicles and 20% commercial car usage.
2.2 Bridge Location & Environment

The water embodied by Pitt Water is known as a semi-estuarine coastal embayment that is tidal, but is predominantly fed by the Coal River. Water velocities have been recorded at 0.17m/s. An analysis of Cationic and Anionic species was conducted on a sample of water collected at the Western Abutment and are shown in Table 2.1. These results are consistent for seawater, and Chloride and Sulphate levels are also consistent with DIER tests conducted on October 27, 1997 [110].
2.3 The Sorell Causeway Bridge: A Historical Perspective

The bridge is located in a cool climate with temperatures averaging $12.2^\circ C$ in winter to $22.1^\circ C$ in summer. Per year on average, the number of days where the temperature is less than $0^\circ C$ is 3.5 days, and above $30^\circ C$ is 6.3 days. The average rainfall is approximately 515mm per year. The bridge is also known to be subjected to strong, prevailing winds from the north-west and south-west. Where winds are strong and tides are high, the bridge is closed for safety purposes due to wave splash over the bridge deck. Due to these conditions and tidal movements, the underside of the superstructure (i.e. precast beams) are continuously being wetted and dried, with only external beams exposed to fresh water from precipitation.

Table 2.1: Concentration of ionic species in seawater taken from Pitt Water Estuary

(a) Cationic Species

<table>
<thead>
<tr>
<th>Cations</th>
<th>Lithium</th>
<th>Sodium</th>
<th>Ammonium</th>
<th>Potassium</th>
<th>Magnesium</th>
<th>Calcium</th>
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</thead>
<tbody>
<tr>
<td>Amount (mg/L)</td>
<td>0.10</td>
<td>12820</td>
<td>-</td>
<td>518</td>
<td>1159</td>
<td>511</td>
</tr>
</tbody>
</table>

(b) Anionic Species

<table>
<thead>
<tr>
<th>Anions</th>
<th>Nitrite</th>
<th>Nitrate</th>
<th>Fluoride</th>
<th>Bromide</th>
<th>Phosphate</th>
<th>Chloride</th>
<th>Sulphate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amount (mg/L)</td>
<td>-</td>
<td>-</td>
<td>3.85</td>
<td>61</td>
<td>-</td>
<td>18255</td>
<td>2515</td>
</tr>
</tbody>
</table>

2.3 The Sorell Causeway Bridge: A Historical Perspective

In the mid 1800’s, the link between Hobart and Port Arthur became increasingly important. To make this journey, however, required travellers to climb aboard small ferries or make a long 40km journey which detoured through Richmond on rutted, dirt tracks. A simpler route would be to construct a series of causeways over the Pittwater estuary saving 20km off the traditional journey, and thus the idea of the Sorell Causeway was born.

---

1This historical account of the Sorell Causeway Bridge has been derived from Bowes [61], Coombs [100], DIER project documentation [110] and Harris and Smith [147]
2.3 The Sorell Causeway Bridge: A Historical Perspective

In the late 1840’s, Sir William Denison began to develop plans to construct a bridge and causeway across the estuary. A number of contractors were employed to construct the causeway, however many found the task difficult and abandoned the works. The bridge was finally completed by the company Oldham and Helmer. Its construction took almost 8 years, supposedly with the assistance of convict labour, and cost twice as much as the original tender, more than £27,000. A toll bar was installed at Midway Point, perhaps to recoup the costs of construction. The original bridge was entirely timber and the causeway a mixture of clay and sandstone rubble. It was opened to the public by Governor Charles Du Cane on June 19, 1874. An overview of the original structure is shown in Figure 2.2. Due to the limited service life of the timber components, this bridge was replaced in its entirety with another timber structure in 1906. It also required extensive maintenance works approximately every 10 years, which proved to be expensive.

![Figure 2.2: Overview of the original Sorell Causeway Bridge from Midway Point, early 1900’s (courtesy of the State Library of Tasmania)](image)

In the late 1940’s, inspectors identified that the bridge was in a poor condition, with timber piles and decking boards found to be in a severely decayed state. In some instances, some of the piles had decayed to the point where they were “floating”, and not providing
any structural support. These alarming observations combined with the fact that main-tenance was frequent and expensive prompted the decision to replace the entire bridge once more. A new alignment was chosen and the Tasmanian State Government decided to replace the bridge with a more permanent and lower maintenance structure. After much discussion and consultation, a precast, post-tensioned structure was decided upon in the early 1950’s. This was a relatively new design and construction technique in Australia; indeed, whilst the prestressing phenomenon was well established in Europe (especially in France and Germany where the work was pioneered), the benefits were only just being realised in America and Australia. For example, the first post-tensioned structure built in America was the Walnut Lane Bridge in Philadelphia, Pennsylvania in 1951 [326], a structure very similar in design and fabrication to the Sorell Causeway Bridge.

The Sorell Causeway Bridge, however, was not the first prestressed structure in Australia at that time. The first pre-tensioned bridge was built in June 1953 across Mittagong Creek in Bowral, NSW. The first post-tensioned structure was built later that same year across Piper’s Creek near Guthega Power Station NSW [161]. The first pre-tensioned bridges in Tasmania were built by the Hydro Electric Commission (the first being over Dee Lagoon spillway in 1955 [97]), the Sorell Causeway Bridge represented the first precast, post-tensioned bridge. In fact, it is claimed that it was the first of its kind in Australia, being the longest precast, prestressed bridge of the day [100]. The use of prestressed, precast piles was, however, the first known application of this technique in Australia.

Design work on the bridge commenced around 1954. This was carried out by the Department of Public Works (DPW) in collaboration with Prestressed Concrete (Australia). The initial design focussed on the use of precast, post-tensioned I-beams with or without a composite deck slab. The motivating factor for this choice is not known, however design advice was sought from consultants in Great Britain regarding the then fashionable Freyssinet prestressing method. John Holland & Co. Pty. Ltd. was awarded the construction contract. They submitted an alternate design which was later accepted for construction. It involved post-tensioned, precast T-Beams with no composite slab. This design, prepared by R. Milton Johnston of Melbourne, is discussed further in Section 2.4. The manufacture of the precast beams was sub-contracted to Humes Ltd., who also
2.4 Design Specification of Post-tensioned Beams

designed and fabricated the prestressed piles [100].

A precast yard was established at Midway Point close to the bridge site, where a concrete batching plant was installed and the beams were prefabricated, as there is a lack of documentation pertaining to construction activities. This was to reduce excessive handling and transportation of the beams prior to installation. The piles were fabricated at a different location. The first beams were cast on February 6, 1956, and it is thought that precasting activities ceased around mid-December 1956. Curing, post-tensioning, and grouting activities of individual beams took place in the precast yard (details of which are discussed in Section 2.4). Once the grout had sufficiently cured, the beams were transported to site and placed side-by-side, leaving a 19mm gap between beam diaphragms. The gaps between were then dry-packed with a sand-cement mixture, ensuring the diaphragm ducts were not contaminated or blocked. High-strength strands were then threaded through the transverse ducts, post-tensioned and grouted.

Coombs recalls that the construction started from Midway Point, working its way towards the Hobart abutment [100]. However in inspecting the beams after demolition, taking note of their span number and unique beam number, it is suggested that the opposite is true, i.e. construction commenced from the Hobart Abutment. The construction photos appear to support this theory. In any case, the Sorell Causeway Bridge was officially opened on April 16, 1957 by the then Premier Robert Cosgrove and Minister for Transport Eric Reece. At its completion, the bridge was 457m long comprising 34 spans, each containing 14 beams. Spans were supported by a series of driven precast, prestressed piles capped with an in situ concrete crosshead. Concrete bridge furniture such as fencing or kerbing was either precast or cast in-place. Construction photos are shown in Figure 2.3.

2.4 Design Specification of Post-tensioned Beams

A total of 476 precast, post-tensioned concrete “T” beams, each about 13m long, were fabricated for use in the bridge. Figures 2.4 and 2.5 show the plan and cross-sectional details of the beams respectively.
2.4 Design Specification of Post-tensioned Beams

(a) Overview of construction to pier level

(b) Placement of beams

Figure 2.3: Construction of the Sorell Causeway Bridge c. 1956

Figure 2.4: Example of beam cross-section summarising dimensions and reinforcement details
Figure 2.5: Summary of beam information from DIER drawings 509 S-46 and S-47, showing beam dimensions and steel arrangements [110]
2.4 Design Specification of Post-tensioned Beams

Beams were cast using steel forms and external vibrators, with strict geometric tolerances being imposed due to constrictive site requirements. Mild steel cages were assembled off-site and spot-welded to maintain rigidity before transporting to site [147]. It is unclear how the post-tensioning ducts were cast, however it is thought that rubber tubes were parabolically draped through the beam and inflated prior to concreting [185, 287]. This theory is supported by Zollman et al. [326], which discusses the use of rubber cores in thin-webbed sections for the construction of the Walnut Lane Bridge. An example is shown in Figure 2.6. No physical duct was installed into the voids post-casting. Once the beams had set, the tubes were deflated and removed, and the beams relocated to a different location to cure until the concrete had reached 5000psi (or 34MPa). In the summer, curing was achieved by wrapping the beams in Sisalcraft (insulation paper) and storing bags of wet sand with the cloth [147] (Figure 2.7).

After curing, prefabricated 0.2” (or 5.1mm) cold-rolled prestressing wire cages were threaded through the longitudinal ducts. Whilst these details are not specified on the drawings, a bridge of similar age and design observed by Vogel [309] provides further detail. Strands were arranged around a central guiding spiral, similar to tie wire. A diagram of the finished cage is shown in Figure 2.8.
Once drawn through the duct, wires were stressed one at a time using a proprietary Freyssinet jack. Pressure grouting then took place and a unique number was assigned to each beam (Figure 2.9). Problems were experienced by the sub-contractor in the prefabrication of the beams as the tight geometric tolerances and steel fabrication quantities generated significant difficulties [147, 185]. Drawbacks also occurred with the fabrication of the initial precast beams due to insufficient bolts between formwork shutters, a problem later rectified with the use of additional bolts. Coombs recalls that 28 beams were condemned due to defects, and it was alleged that some beams may have been repaired and used in the bridge without the consent of DPW.

(a) Preparing to grout longitudinal ducts
(b) Completing post-tensioning activities

Figure 2.9: Post-tensioning detail of beams
2.5 Deterioration of the Bridge

As stated in the previous section, issues were experienced by the sub-contractor in the fabrication of the beams. This is to be expected, as this technology was relatively new and untested by authorities and consultants in Australia. Whether these issues have translated to corrosion-related defects it is unclear but worth noting in this context.

The first instance of corrosion was noted not long after the bridge had been completed. Approximately 18 months after completion, the soffits of a number of pier crossheads required repair due to the presence of small steel punchings. These punchings were used as part of a counterweight in the on-site crane used for installation activities. The box containing the punchings had a small leak and many of the punchings fell into the concrete pour, contaminating the crossheads [185].

Limited information relating to bridge inspection records was found between the 1970’s and late 1980’s. To the author’s knowledge, the first documented case regarding structurally related corrosion is in 1978, where a file note records details from a bridge inspection undertaken on 18 January that year. Observations were made from land. Spalling was observed along the tops of piles from the easternmost pier, and as recorded in the inspector’s notes [110]: “This condition is not new and has been reported on previously”. He had also noted that “no significant spalling was observed on crossheads”.

A further matter of concern was raised in a letter sent to DPW on November 17 1976 by G.P. Cook [98], who was the Project Manager for John Holland during the construction of the bridge. He advised that Calcium Chloride had been used as an accelerator in the manufacture of the precast beams (a substance now banned from structural concrete applications). Cook stated that the additive may not be an issue for the cement grout in the ducts, stating “to the best of my knowledge, the cement grout used to fill the voids around them did not contain calcium chloride”. He recommended that a detailed investigation be carried out on the beams, with special attention to the prestressing strands and anchors.

These observations have been summarised from DIER project documentation [110]
2.5 Deterioration of the Bridge

Whether the DPW conducted an inspection immediately after this letter was received is unclear; however on November 23 1979 an under-bridge inspection was conducted, with the specific intent of examining the piles and deck. Significant spalling and rust staining was observed on piles and some crossheads. No specific mention was made of any cracking or corrosion-related defects relating to the post-tensioning strands, however instances of spalling across the corner of Beams 8 and 9 over Pier 4 to the first diaphragm on Span 5 were noted. No photos were taken of this defect, so the exact extent and characterization of this defect remains unknown. The deterioration of grout from anchor blocks on beams from Spans 13 and 14 was noted also.

On September 30 1987, a facsimile was sent to the Minister of Main Roads regarding the condition of the Sorell Causeway Bridge. It was stated that the bridge was thoroughly inspected on a regular basis and that “no deterioration of structural significance is known”. Nevertheless, repairs were made on piles and crossheads in early 1982 with inspectors citing “severe chloride attack” [110].

In February 1993, a detailed underbridge inspection of the entire bridge was conducted in which the inspector found numerous corrosion-related defects across the beams. In this report, it was observed that 55 beams exhibited cracks and delaminations along the bottom corners of the beams between the first diaphragm and the end of the beam. A number beams with delaminations across the side face of the beams between the first and second diaphragms, stating “delamination appears to start as a horizontal crack 170mm up from soffit of beam. Some beams also have a longitudinal crack running down the centre of the beam soffit”. A total of 10 beams were noted to have such defects, including Beam 9 from Span 5 and Beams 12 and 14 from Span 17 (numbered 1 to 14 starting from the north). The crack width of Beam 9 was approximately 0.5mm. No crack widths were given from Beam 12 of Span 17, but some spalling over shear ligatures is also observed. It is not clear from the data whether these cracks are located within the web or the base flange, or the appearance of its trajectory.

In 1994, Façade Technology was contracted to carry out a detailed condition survey of the bridge [118]. In the report, piles were noted to be in a poor condition and beams and cross-heads in a fair to poor condition; the majority of defects related to corrosion.
2.5 Deterioration of the Bridge

The first official observation of longitudinal web cracking in Span 17, stating “we are concerned about the web cracks observed between grids 17 and 18... this is a one off occurrence which requires immediate repairs”. Photos 31 and 32 in their report show that these cracks are substantial in size. This appears to be the only reference to web cracking and nowhere near the number of defects identified in the 1993 inspection report. No explanations were given regarding the evolution of this type of cracking.

In November 1996, an additional underbridge inspection identified ten beams with instances of longitudinal web cracking, although crack widths and lengths are not documented. Further instances of spalling and corrosion related defects were also noted, with the inspector stating “on comparing beam distress [to that observed] by Façade Technology there is a considerable increase in the number of beams showing web cracking and spalling”. A brief visual inspection was conducted by Sinclair Knight Merz on behalf of Infratech Systems & Services in 1997 as part of a structural analysis and load testing exercise of the bridge [157]. In it, the report states that Spans 1, 5, 6, and 7 were in a poor to satisfactory condition, with a number of beams suffering spalling and longitudinal cracking at beam ends. Instances of longitudinal cracking in varying stages were observed in beams from Spans 9, 10, 12, 13, 15, 17, 21, 23, and 25. It did not specify which beams are affected and the severity of the defects. It does state, however, that longitudinal cracking appeared to follow the trajectory of the lower prestress cable, and that it occurred towards the supported end of the beam. The loss of reinforcement section and the condition of the prestressing strands from the transverse ducts was also called into question. Despite this documented evidence, however, Ministerial correspondence cited in July 1998 stated that the bridge was still considered to be in a serviceable condition and that “the bridge is not in danger of failure”.

In 1999, two additional longitudinal cracks were identified on beams from Spans 4 and 5 during a routine underbridge inspection. These cracks were severe and traversed almost the full length of a bay. The inspection report noted crack widths of 3mm and 5mm in Beam 14 from Span 4 and Beam 12 from Span 5 respectively. These cracks were not identified or flagged as serious defects in previous inspections. Figure 2.10 shows a typical example of a longitudinal web crack from Span 5. Observe the trajectory of the crack.
as it approaches the base flange in the background. A crack gauge at Point A is also observed, of which crack monitoring results were obtained (see Section 2.6.1).

Figure 2.10: An example of a significant longitudinal web crack that has developed in a beam from Span 5 whilst in service

By 2000, 51 longitudinal web cracking sites had been identified, in comparison to one official case in Span 17 in 1994. The cracks were observed on 31 beams, of which 18 beams exhibited simultaneous parallel cracks on both sides of the webs. Figure 2.11 shows a plot constructed by DIER showing the increase in the number of beams affected by this form of cracking over time. Note that observed cases tripled between 1993 and 1996, and again between 1996 and 1997. A steady increase is then observed between 1997 and 2000.

Due to the increasing occurrence of the longitudinal cracking along with condition findings from several consultancy reports (Section 2.6), an independent review was conducted in early 2000 in order to determine the risks surrounding the possibility of the bridge becoming unserviceable or collapsing [189]. This review not only highlighted the poor condition of the bridge (especially in relation to the longitudinal and transverse prestressing strands), but recommended that the worst span (Span 17) be replaced and speed
restrictions be enforced to reduced dynamic impact loads. However there were still concerns surrounding the full extent and severity of the strand corrosion, and as the bridge represented a significant economical and social risk if it collapsed, the Tasmanian State Government made the executive decision to replace the bridge effective immediately.

A contract for the replacement of the bridge was awarded to John Holland Pty. Ltd. in 2001, the same contractor who had built the original Sorell Causeway bridge, and site works began in September that same year. On December 14 2002, the new Sorell Causeway Bridge, renamed McGee’s Bridge in honour of the late Dr. Rod McGee, was opened to the public. The bridge replacement cost approximately A$20 million. The old bridge was demolished in early 2003 (Figure 2.12) due to concerns surrounding its safety and future cost implications if a bridge duplication was required. At the time of demolition, the old Sorell Causeway Bridge was only 45 years old, and it is thought to be one of the first major post-World War II concrete structure to be demolished in Australia.

Whilst the bridge was still in service, a recommendation from the 2000 risk assessment report [189] was to remove Span 17 in its entirety and subject it to detailed testing (including load testing) in order to gain better understanding of the performance of the structure, especially in light of the numerous investigations carried out. With the decision to replace the bridge, the immediate need for such testing was negated. However, the
opportunity to further this research was realised by the University of Newcastle and DIER, and funding was sought and approved from the Australian Research Council. Subsequently, three beams were salvaged from the bridge after demolition and transported by sea and road to the University of Newcastle for further detailed testing. They were selected on the basis of condition (ranging from good to very poor) as summarised in Table 2.2. Further detail regarding the beam specimens is reported in Appendix A. These three beams comprise the basis of the work contained herein.

Table 2.2: Beams selected for the present investigation

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Beam Cast #</th>
<th>Casting Date (1956)</th>
<th>Location</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>B17/4</td>
<td>254</td>
<td>17 Aug</td>
<td>Span 17, Beam #11 (or #4)</td>
<td>Good; minor spalls over ligatures</td>
</tr>
<tr>
<td>B17/3</td>
<td>256</td>
<td>18 Aug</td>
<td>Span 17, Beam #12 (or #3)</td>
<td>Poor; severe longitudinal web crack</td>
</tr>
<tr>
<td>B118</td>
<td>118</td>
<td>18 May</td>
<td>Unknown; Between Spans 6 and 8</td>
<td>Reasonable; smaller longitudinal crack; spall at end bay</td>
</tr>
</tbody>
</table>
2.6 Review of Pre-Demolition Project Data

After the first official crack was identified by Façade Technology in 1994, an extensive program of inspections and diagnostic tests were conducted on the Sorell Causeway Bridge, in an attempt to establish the condition and remaining service life of this bridge. The majority of these tests were carried out by consultants up until the bridge was decommissioned, at great cost to DIER. A variety of opinions and assessments were presented, which has no doubt added to the confusion in assessing the true condition of the bridge. The investigations are listed in chronological order in Table 2.3, and the key findings of each are briefly reviewed in the following paragraphs.

2.6.1 In-House Inspection Records and Defect Monitoring

Very little documentation is available in project records regarding bridge inspections conducted prior to 1990. After this time, a systematic inspection program for bridge structures was established by DIER and more regular inspection data is available. Observations additional to those made in Section 2.5 are discussed in this section. Detailed and colour photographs are available from the later inspections, which show rust staining, cracking and concrete loss on the piles and crossheads. Limited rust staining and severe spalls are observed on some beams; in some areas, these spalls extend to the depth of prestressing strands.

Between 1998 and 2000, DIER inspectors kept a detailed record of defects for each beam in the bridge. Individual beams were allocated into bays as determined by the diaphragms (refer to Appendix A for nomenclature), and defects were classified as cracking (normal), spalling or longitudinal web cracking with respect to their location. These results were reviewed and combined to illustrate the location of defects across the bridge, shown in Figure 2.13. Each beam in each span is represented and defects are noted in accordance with their location along the beam (per bay). It was noted that the majority of defects occurred within the first half of the bridge. A significant proportion of the defects occurred along the southern-most beams and towards the centre of the span, with several web cracks observed directly beneath wheel paths. Higher frequency of defects (specifically cracking) existed around the beam ends, in particular the anchor pier at Pier 5.
2.6 Review of Pre-Demolition Project Data

Table 2.3: List of investigations carried out on the Sorell Causeway Bridge prior to decommission

(a) In-house investigations

<table>
<thead>
<tr>
<th>Date</th>
<th>Type of Report</th>
</tr>
</thead>
<tbody>
<tr>
<td>1999 - 2001</td>
<td>Generic annual bridge inspections</td>
</tr>
<tr>
<td>1993 - 2000</td>
<td>Annual underbridge inspections</td>
</tr>
<tr>
<td>May 1997 (Materials &amp; Research Division)</td>
<td>Chloride profiles</td>
</tr>
<tr>
<td></td>
<td>Cement Contents</td>
</tr>
<tr>
<td></td>
<td>Carbonation</td>
</tr>
<tr>
<td></td>
<td>Cover Survey</td>
</tr>
</tbody>
</table>

(b) Consultancy Reports

<table>
<thead>
<tr>
<th>Date</th>
<th>Company</th>
<th>Aim of Investigation</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 1992</td>
<td>Hydro Electric Commission</td>
<td>Concrete properties</td>
</tr>
<tr>
<td>March 1994</td>
<td>Façade Technology/Structural Systems/Solomon Corrosion</td>
<td>Detailed inspection Non-destructive testing Chloride and carbonation profiles Concrete properties</td>
</tr>
<tr>
<td>October 1996</td>
<td>Remedial Engineering Group</td>
<td>Cathodic protection trial (piers) Coatings trial (beams)</td>
</tr>
<tr>
<td>February 1999</td>
<td>Infratech Systems &amp; Services</td>
<td>Visual Inspection Structural Analysis of the bridge Load testing of Span 1 Load testing and long-term monitoring of Span 17</td>
</tr>
<tr>
<td>May 2000</td>
<td>University of Newcastle (TUNRA)</td>
<td>Risk assessment of the bridge Review of previous consultant reports</td>
</tr>
</tbody>
</table>
2.6 Review of Pre-Demolition Project Data

Figure 2.13: Visual summary of defect locations across bridge recorded in 1998 - 2000 DIER inspections.
2.6 Review of Pre-Demolition Project Data

Summaries of these results are shown in Figure 2.14. For the whole bridge, a total of 238 defects were observed; almost half of those defects were corrosion-related spalls, whilst the other half related to instances of cracking. Of the latter, approximately a third of these were longitudinal web cracks (Figure 2.14a). On average per beam, the two end bays and the eastern-most internal bay (or Section 1) show the greatest number of defects in relation to other bays (Figure 2.14b).

Figure 2.14: Statistical representation of defects across the bridge

Figure 2.14c shows the number of beams per span that exhibit some form of defect, showing a trend of a greater number of defects found on beams towards the western side of the bridge. This confirms what was observed visually in Figure 2.13. Figure 2.14d shows the total number of beams with observed defects with regards to its lateral position across the bridge. In summary, the beam containing the most number of defects was Beam #14, or the southern-most beam. A greater number of defects were also observed on internal beams closer to the northern face of the bridge. Unfortunately, the data recorded did not differentiate between the number of defects observed or the northern or southern faces of the beams.
2.6 Review of Pre-Demolition Project Data

Figure 2.14: Statistical representation of defects across the bridge (cont’d)

A number of beams were selected for crack monitoring whilst the bridge was still in service. The crack gauges were installed in 1999 and were monitored approximately monthly. The beams selected for monitoring were Beam #12 from Span 17 (or known as Beam 17/3 for the present study), Beam #14 from Span 4 and Beam #12 from Span 5.
All cracks showed some degree of growth over the 21 month period, ranging between 0.5 and 1.5mm, with Diaphragm (iv) showing the most significant signs of growth. The crack in Beam #12 from Span 17 (Beam 17/3) had been monitored for growth since the Façade Technology report in 1994 [118], however documentation does not specify which face these measurements were taken from. Observations of crack width measurements are made in Table 2.4. The crack had grown substantially from the time it was officially noted in 1994 to the last informal DIER inspection in 2000.

Table 2.4: Record of crack width growth on Beam #12 from Span 17 (i.e. Beam 17/3)

<table>
<thead>
<tr>
<th>Date</th>
<th>Inspector</th>
<th>Crack Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>1994</td>
<td>Façade Technology</td>
<td>2-3mm (approximate)</td>
</tr>
<tr>
<td>1997</td>
<td>Infratech</td>
<td>10mm</td>
</tr>
<tr>
<td>1999</td>
<td>DIER</td>
<td>5mm++ at base flange (approximate)</td>
</tr>
<tr>
<td>2000</td>
<td>DIER (informal)</td>
<td>15mm (approximate)</td>
</tr>
</tbody>
</table>

Façade Technology provided the first condition assessment in their 1994 report [118], stating that a total of 32 beams had visual defects, 6 with spalls and 7 with cracks mostly in the base flange. Only one beam was officially identified with longitudinal web cracking (beam within Span 17), discussed previously in Section 2.5. It is difficult to correlate the location of defects with specific beam numbers and locations noted in the consultancy reports. However it is thought that the beam containing the longitudinal crack is the same beam discussed in later consultancy reports (Beam 17/3). It was the opinion of Façade Technology that the majority of the bridge beams were “visually sound” with a small number of defects observed on western end of the bridge. The report’s primary concern was the longitudinal web crack, recommending structural repairs (such as epoxy crack injection). They also recommended patch repairs and investigations into the application of a Cathodic Protection System or a protective water-proof coating to arrest corrosion.
2.6 Review of Pre-Demolition Project Data

2.6.2 Non-Destructive Testing

The report submitted by Façade Technology in 1994 contains the majority of results in relation to insitu non-destructive testing carried out on the bridge. These included surveys of cover, half-cell potentials, and concrete resistivity. A brief review of these surveys is discussed in the following paragraphs.

2.6.2.1 Covermeter Survey [110, 118]

Covermeter surveys were conducted by both Façade Technology in 1994 [118] and DIER’s Materials and Research division (M&R) in 1997 [110]. The survey conducted by Façade Technology was carried out on four beams from Spans 6, 12, 17 and 32 (Grids 28/29, 22/23, 17/18, and 2/3 respectively), which is shown in Figure 2.15. The cover to shear ligatures and prestressing strands was measured on web faces; additional cover readings were taken for the shear ligatures along the beam soffit.

![Figure 2.15: Covermeter survey locations conducted on the Sorell Causeway bridge beams whilst in service](image)

An extract from Façade’s report on covermeter results is shown in Figure 2.16. In summary, web cover to ligatures ranged between 23mm and 55mm, with the lowest covers of 23 - 35mm recorded across beams from Spans 17 and 12. Beam soffit covers were less on average, measuring between 23 and 30mm. Covers to strands were not less than 40mm. There is little information to specify the location or extent of the survey, as well as the total number and standard deviation of results. The report stated that there was
2.6 Review of Pre-Demolition Project Data

general compliance between cover specified on the drawings and those measured on site. However, in light of the small sample size, this statement may not be completely accurate.

In contrast, results from the covermeter survey conducted by M&R in 1997 reveal low and variable covers. The survey was limited to five beams from Span 17 for this instance (as shown in Figure 2.15), showing the average and standard deviation of covers measured over shear ligatures and prestressing strands (see Figure 2.17). The survey included Beam 17/3. For shear ligatures, covers as low as 18mm on average were detected, which were covers typically lower across the northern face of the beams than the southern face. Standard deviations ranged between 0.8 and 9.8mm. Beam 17/3 recorded maximum and minimum average covers of 38mm and 30mm respectively. Soffit covers to shear ligatures ranged between 24 and 53mm for all beams. Covers measured for the prestressing strands generally agreed with those measured by Façade Technology; covers generally exceeded 36mm with the exception of one beam with an average cover of 30mm measured across the northern face of the beam.
2.6 Review of Pre-Demolition Project Data

![Table: I Beams: Average Depth of Cover and Depth of Carbonation Results]

<table>
<thead>
<tr>
<th>Element</th>
<th>Core</th>
<th>Location</th>
<th>Depth of Cover (millimetres)</th>
<th>Depth of Carbonation</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>I Beams</td>
<td>-</td>
<td>Centre</td>
<td>Web ligs: F - 47-65</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Web cables: 42, 45, 72</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt reading: 30-38</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>IG8</td>
<td>Grid 12-18 Centre</td>
<td>Web ligs: F - 37-39</td>
<td>Outside 1.6</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: S - 23</td>
<td>Inside 0.21</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Grid 12-18 North end</td>
<td>Web ligs: F - 25-42</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Web cables: F - 48</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: S - 23-30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>IG9</td>
<td>Grid 22-29 Near centre</td>
<td>Web ligs: F - 41-43</td>
<td>Outside 2.5</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: F - 28</td>
<td>Inside 2-12</td>
<td></td>
</tr>
<tr>
<td></td>
<td>IG10</td>
<td>Grid 22-29 South end</td>
<td>Web ligs: F - 29</td>
<td>Outside 1.7</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Web Cable: F - 41</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: S - 27</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>IG11</td>
<td>Grid 26-29 Centre</td>
<td>Web ligs: F - 40</td>
<td>Outside 2.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: S - 24</td>
<td>Inside 0.19</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Grid 26-29 North end</td>
<td>Web ligs: F - 40</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Web cable: F - 45</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bolt flange: S - 30-47</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.16: Summary of 1994 covermeter readings
(extract from Façade Technology Report [118])

![Diagram: Beam No. Vertical Strands: Pre-stress Strands: Underivide Strands]

<table>
<thead>
<tr>
<th>Beam No</th>
<th>North side face:</th>
<th>South side face:</th>
<th>Underivative face:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - x:</td>
<td>a: 33 3.1</td>
<td>18 4.1</td>
<td>30 2.6</td>
</tr>
<tr>
<td>31 - x:</td>
<td>37 9.8</td>
<td>30 8.3</td>
<td>-</td>
</tr>
<tr>
<td>7 - x:</td>
<td>34 3.5</td>
<td>23 4.5</td>
<td>36 2.7</td>
</tr>
<tr>
<td>12 - x:</td>
<td>36 6.4</td>
<td>30 6.2</td>
<td>43 2.2</td>
</tr>
<tr>
<td>14 - x:</td>
<td>40 3.8</td>
<td>35 5.0</td>
<td>43 5.3</td>
</tr>
</tbody>
</table>

Figure 2.17: Summary of 1994 covermeter readings
(extract from DIER M&R Report [110])
2.6 Review of Pre-Demolition Project Data

2.6.2.2 Half-Cell Potential Survey [118]

Half-cell potential surveys were conducted by Solomon Corrosion Control Services for Façade Technology [118]. Only four beams were surveyed, one each from Spans 7 and 33 (Grids 27/28 and 1/2 respectively), and two from Span 17 (Grid 17/18) (see Figure 2.18 for survey locations). Beam 17/3 is one of the beams tested. Surveys were conducted using a Copper/Copper Sulphate reference half cell over a 250mm square grid. Vertical beam faces were surveyed, such as the web and base flange surfaces. Connection to exposed reinforcement due to spalling or coring activities completed the electrical connection of the half-cell.

Potentials ranged between of 0 to -500 mV. Generally, potentials were increasingly negative towards the base of the beams. The most negative potentials were measured along the lower edge of a beam from Span 7, with readings consistently below -400 mV. The most detailed surveys were conducted on Bay 5 of Beam 17/3 (the site of the worst cracking) and an end bay from Beam 17/5, of which the results have been replicated visually in Figures 2.19a and 2.19b respectively. Readings from Beam 17/3 show potentials generally more negative than -200 mV, with a minimum of -410mV observed along the lower edge of the southern web at the centre of bay. Half-cell potentials more negative than -350mV are more likely to indicate a region of corrosion, according to limits established in the literature [43] (Chapter 4).

![Figure 2.18: Half-Cell Potential survey locations conducted on the Sorell Causeway bridge beams whilst in service](image)
2.6 Review of Pre-Demolition Project Data

(a) Results for Bay 5 of Beam 17/3 ("Western" face, in mV with respect to a Copper-Sulphate Electrode)

(b) Results for Bay 6 of Beam 17/5 ("Eastern" face, in mV with respect to a Copper-Sulphate Electrode)

Figure 2.19: A visual summary of the Façade Technology half-cell potential surveys
2.6 Review of Pre-Demolition Project Data

2.6.2.3 Concrete Resistivity Survey [118]

Very little data relating to concrete resistivity was found in all consultancy reports. Façade Technology appears to be the only consultant to have measured the resistivity of the piles, crossheads and beams [118]. There is no detailed description regarding the test methodology and test locations but a Wenner 4 Pin resistivity test was used for data collection. In total, only seven resistivity readings were obtained, five from a number of piles and one each from a crosshead and a beam (see Figure 2.20 for the approximate location of the test conducted on the beam). The report states that the concrete surface was found to be very hard and that masonry nails were not able to be driven into the concrete at all designated test sites.

The resistivity value for the beam (thought to be Beam 17/3) was 0.85 kΩ.cm; according to the literature, this value indicates that the concrete will offer little (if any) resistance in the event of reinforcement corrosion [243]. The report states that these results are probably not representative of the overall resistivity of the concrete and are likely to be unreliable due to “insufficient electrode penetration into the concrete, coupled with a low resistivity chloride surface”.

Figure 2.20: Resistivity survey location conducted on the Sorell Causeway bridge beams whilst in service
2.6 Review of Pre-Demolition Project Data

2.6.3 Destructive Testing

Several “destructive” tests were carried out over the beams, with the predominant focus on chloride profiles. Carbonation depths, concrete properties and sulphate contents were also documented in several reports. A review of this data is now presented.

2.6.3.1 Chloride Profiles [118, 277, 278]

Two reports contain independent information regarding the chloride profiles of various beams. These include the 1994 Façade report [118] and the 1997 Taywood Engineering review report [277, 278] which incorporates test information obtained by DIER’s M&R division [110].

Façade’s report reflects chloride profiles obtained from in situ 50mm diameter cores taken through the webs of external beams, typically from the “northern” end of the beam although specific locations are not detailed. Approximated test locations are shown in Figure 2.21. No results were obtained for internal beams due to access constraints. A total of 4 cores were retrieved from Spans 6, 12, and 17 (Grids 28/29, 22/23, and 17/18 respectively), which were sliced into 15-20mm approximate increments and treated in accordance with the relevant standards [33, 72]. It is not clear if the entire core was used and the results presented are averages. Chlorides were expressed as a percentage of both cement and concrete by mass.

![Figure 2.21: Chloride profile test locations conducted on the Sorell Causeway bridge beams whilst in service](image-url)
Results are shown in Figure 2.22. All chloride results were elevated, with a minimum result of 0.31% Chloride by weight of cement (\(\%Cl^- \text{(cement)}\)) or 0.06% Chloride by weight of concrete (\(\%Cl^- \text{(concrete)}\)) - this was greater than the minimum chloride threshold recommended in the literature (Chapter 5). The maximum concentration recorded was 1.16\(\%Cl^- \text{(cement)}\) on the surface of a beam from Span 6. Concentrations appeared to decrease with increasing depth, but were still elevated at rebar and prestressing levels. Even higher chloride concentrations were observed across the piles and crossheads, with maximums of 2.72\(\%\) and 4.13\(\%\) \(Cl^-\) (cement) observed respectively. However this appeared superficial for the piles, as concentrations dropped significantly with increasing depth.

Façade Engineering stated that a direct correlation between chloride concentration and cement content is evident, which may be a hasty conclusion due to the small number of samples obtained. It was stated that although elevated concentrations exist at reinforcement level, the “generally good condition” of the beams can be attributed to their “dry condition”. This is a surprising conclusion since these beams were frequently wetted and dried due to tidal movements, making this an inaccurate claim.

![Figure 2.22: Summary of 1994 chloride profiles](image)

(extract from Façade Technology Report [118])
The lack of detailed location information and sample sizes from the Façade report prompted further chloride tests on the beams. These additional tests were carried out by M&R on beams from Span 17 (the same as those surveyed for cover) and subsequently reviewed by Taywood Engineering [277, 278]. Beams selected by DIER for sampling are shown in Figure 2.21, which includes Beam 17/3. Samples were retrieved at 1.5m and 4.0m from a beam end using a hammer drill at set depths, collecting the concrete powder in sealed plastic bags. Typically, samples were obtained to the depth of prestressing tendons on both north and south faces and to the depth of reinforcement along the beam soffit, with set sample locations shown in Figure 2.23. All samples were expressed as a chloride percentage by mass of concrete and are shown in Figure 2.24.

Figure 2.23: Generic location of chloride samples
(extract from DIER M&R Report [110])

Taywood’s interpretation of the 1997 chloride levels agree with the sentiments expressed in the Façade report. All chloride concentrations again exceeded the threshold requirements recommended in the literature [2, 51], with a minimum concentration of 0.08%Cl\(^-\) (concrete). The maximum concentration of 0.63%Cl\(^-\) (concrete) was recorded on the surface of the north face of Beam 17/3, adjacent to Diaphragm (iv). Concentrations were greater across northern faces compared to southern faces, especially in relation to superficial chloride levels. Levels remained elevated with depth and appeared consistent when comparing web and soffit measurements.
Taywood Engineering have interpreted the results to indicate that there is “widespread corrosion initiation associated with both the ligatures and the strands”, due to the elevated concentrations. They also suggested that further investigations be conducted to determine whether localised corrosion had commenced in conjunction with the high chloride levels.

<table>
<thead>
<tr>
<th>Beam No(1)</th>
<th>Sampling location(2):</th>
<th>Longitudinal position 1: 5m (3)</th>
<th>Longitudinal position 4: 8m (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0-10mm 10-30mm 30-50mm</td>
<td>0-10mm 10-30mm 30-50mm</td>
</tr>
<tr>
<td>1</td>
<td>North side face:</td>
<td>0.27 0.18 0.05</td>
<td>0.19 0.14 0.46</td>
</tr>
<tr>
<td></td>
<td>Underside:</td>
<td>0.30 0.23 0.23</td>
<td>0.32 0.22 0.16</td>
</tr>
<tr>
<td></td>
<td>South side face:</td>
<td>0.15 0.12 0.08</td>
<td>0.11 0.11 0.12</td>
</tr>
<tr>
<td>3</td>
<td>North side face:</td>
<td>0.43 0.52 0.37</td>
<td>0.63 0.52 0.44</td>
</tr>
<tr>
<td>7</td>
<td>North side face:</td>
<td>0.48 0.29 0.20</td>
<td>0.44 0.38 0.25</td>
</tr>
<tr>
<td>12</td>
<td>North side face:</td>
<td>0.36 0.34 0.30</td>
<td>0.19 0.20 0.15</td>
</tr>
<tr>
<td>14</td>
<td>North side face:</td>
<td>0.26 0.29 0.21</td>
<td>0.23 0.21 0.17</td>
</tr>
<tr>
<td></td>
<td>Underside:</td>
<td>0.30 0.21 0.09</td>
<td>0.30 0.21 0.31</td>
</tr>
<tr>
<td></td>
<td>South side face:</td>
<td>0.39 0.13 0.09</td>
<td>0.46 0.20 0.11</td>
</tr>
<tr>
<td></td>
<td>Underside:</td>
<td>0.36 0.31 0.27</td>
<td>0.25 0.19 0.14</td>
</tr>
<tr>
<td></td>
<td>South side face:</td>
<td>0.42 0.13 0.10</td>
<td>0.22 0.12 0.09</td>
</tr>
<tr>
<td>2</td>
<td>Top flange - underside</td>
<td>0.42 0.20 0.16</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Figure 2.24: Summary of 1994 chloride profiles
(extract from DIER M&R Report [110])

### 2.6.3.2 Carbonation Depths [118, 277, 278]

Carbonation depths were estimated by Façade Technology [118] and Taywood Engineering [277, 278], predominantly in conjunction with the cores obtained for chloride profiling. Façade Technology initially identified areas of deep carbonation on both cross-heads and beams. Depths of between 5 and 19mm were measured on cores obtained from Pier 23.
2.6 Review of Pre-Demolition Project Data

(Grid 11) and Pier 32 (Grid 3) crossheads. Similar depths were measured from internal and external surfaces of beams from Spans 6, 12, and 17 (Grids 28/29, 22/23, and 17/18 respectively), with a maximum of 21mm measured from internal surfaces of beams from Span 17. At these levels, the carbonation front was close to reinforcement depths. Façade state that the deep carbonation front observed on internal beam surfaces “is due to the absence of a coating and their generally dry condition”.

This statement and the overall results obtained by Façade were treated with scepticism by Taywood Engineering. Taywood stated that the depth of carbonation is “particularly concerning” especially in the light that only a limited number of samples were tested. In tests conducted by M&R [110], carbonation levels less than 5mm were found at all test sites, directly contradicting Façade’s findings, with beam #1 corresponding to the same beam cored as part of the 1994 investigations.

2.6.3.3 Materials Properties [110, 118]

Concrete density and cement content were determined by Façade Technology [118]. An average density of 2395 $kg/m^3$ was recorded for precast beams with an average cement content of 507 $kg/m^2$ (or 19.5% by weight of concrete). Cement contents recorded by M&R averaged 17.7% by weight of concrete [110].

2.6.4 Structural Analysis and Load Testing [157]

Part of the concerns regarding the Sorell Causeway Bridge revolved around its structural behaviour under load, especially in light of its condition. DIER was also in the process of reviewing its bridge network in relation to increased mass limits, and due to the age of the bridge its compliance with the current-day codes was questioned. Therefore Infratech Systems and Services (Infratech) was commissioned in 1997 to conduct a structural review of the bridge. This included a theoretical structural assessment model (by sub-consultants Sinclair Knight Merz (SKM)) and load testing of Spans 1 and 17 of the bridge [157].

The structural analysis was based on a finite-element analysis, where the bridge was analysed in accordance with the 1992 Austroads Bridge Design Code (ABDC) [48]. Whole spans were considered in the analysis, incorporating the action of the diaphragms. The bridge was assumed to be in good condition. In summary, the reports states that the
structure was “operating close to capacity”, with the analysis showing that central diaphragm and beams were understrength for the current T44 design load, and did not comply with the ABDC overall. The model was found to be sensitive to dynamic loads and the distribution of loads through the diaphragms.

Load tests were initially conducted on Span 1 to correlate the findings from the structural analysis to the physical response of the bridge in situ. Responses were measured from strategically placed strain gauges (as shown in Figure 2.25) after running a number of test trucks simulating various combinations of the T44 design load. Infratech concluded that the “in-service performance of the bridge is significantly better than that predicted by the analytical assessment”, recommending that the structure has sufficient capacity to support the NRTC Option F B-Double and Semi Trailer, and suggesting that the structure could sustain a T44 design load. It was also stated that shear failure did not pose a significant threat to the stability of the bridge based on the observation of strain measurements in the central diaphragms and beams.

Load testing was also conducted on Span 17 with particular attention paid to cracking occurring on Beam #12 (or Beam 17/3). Strain responses were recorded for load test vehicles initially, with long-term data collected for ambient traffic over 3 months. The location of the strain gauges are shown in Figure 2.26, which also highlight the position of Beams 17/4 and 17/3 which have been examined in the current study. Beam 118 was not load tested whilst in service based on the load test records. Infratech state that Span 17 responded in a similar manner to Span 1. However a review of these results showed that the load response for Beam #12 in Span 1 was 43% “greater” that the equivalent cracked beam in Span 17 when using a “slightly heavier vehicle”. A review of strain data from Span 17 shows that loads induced from the ambient traffic were “slightly higher” than the load test vehicle, and that instances of overloading was evident. Crack movements under ambient traffic load were also measured but were considered to be insignificant, recording a maximum strain of 35με. Infratech thus concluded that Span 17 was capable of supporting a Semi-Trailer and B-Double vehicle, stating that Span 17 is “behaving considerably better than predicted by the analysis” and that the deterioration observed “is not influencing the structural performance of this span”.

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2.6 Review of Pre-Demolition Project Data

Figure 2.25: Placement of strain gauges for insitu load tests across Span 1

Figure 2.26: Placement of strain gauges for insitu load tests across Span 17
2.7 Similar Case Studies

Prior to commencing experimental work, a search of the literature was conducted to identify bridges of similar age, design, and location that have been demolished or rehabilitated. Information was also sought in relation to field investigations and steel condition. These examples are now briefly discussed in relation to their relevance to the present investigation.

**Walnut Lane Memorial Bridge in Philadelphia, USA (1951) [20, 326, 327]**

This bridge was the first post-tensioned structure to be built in the United States. The beams are of similar specification to those from the Sorell Causeway Bridge, with use of inflatable rubber tubes to form the post-tensioning ducts. The use of diaphragms was also a similarity. The bridge was replaced in 1990 due to the evolution of large longitudinal web cracks that appeared to follow the trajectory of the ducts (see Figure 2.27). These cracks were found to be the result of strand corrosion due to water entering the ducts at the end anchorage points from the deck. Voids in the grout were also thought to have contributed to its deterioration.

![Figure 2.27: Overview of the old Walnut Lane Memorial Bridge, showing the development of a longitudinal web crack][1]
Ynys-y-Gwas Bridge in Afan Valley, UK (1953) [321]

This bridge collapsed without warning in 1985 with only the outer edge beams left standing. The bridge is somewhat similar in design to the Sorell bridge, using the Freyssinet post-tensioning system and transverse post-tensioning to stiffen the deck. The reasons behind the sudden collapse was blamed on the severe corrosion of strands at the transverse joint locations. Strands at this location should have been protected by a metal sheath however only cardboard tubes were observed at these locations. High chloride levels (from de-icing salts) were blamed for the corrosion. This example is of interest due to the design of the transverse ducts for the Sorell bridge. The techniques used for the Ynys-y-Gwas bridge investigation have also been useful in planning investigations for the present work.

Horwerstrasse Overpass in Kriens, Switzerland (1954) [308, 309]

This structure was one of the first post-tensioned structures in Switzerland, and utilised the Freyssinet post-tensioning system similar to those observed in the Sorell bridge beams. Various non-destructive evaluation techniques were utilised to assess the condition of the bridge prior to retrieving prestressing strands for inspection. Some strands were found to be corroded. The report concludes that a combination of grout voids, the presence of the guiding spiral, chloride contamination, and water leakage into the ducts had facilitated the corrosion process.

The Bissle Bridge in Connecticut, USA (1957) [252, 253]

Again, this bridge is similar in design to the Sorell bridge. The Freyssinet post-tensioning system was employed, however a metal sheath formed the duct and there are a greater number of tendons. The grout was noted to have a high chloride content. Significant corrosion was found on the strands which did not always correlate to grout voids. Longitudinal web cracks were also observed. Where corrosion had occurred on strands from fully grouted ducts, severe pitting was observed and parts of the strands were noted to have bright, metallic surfaces. No longitudinal web cracking occurred for this scenario. No explanations were offered relating to the latter observations.
2.7 Similar Case Studies

**Post-tensioned concrete bridges, UK (1958-1977) [320]**

Several post-tensioned bridges of similar design to the Sorell bridge are discussed in this report. The predominant focus of this report is on grout voids and whether corrosion had initiated in these areas. It was found that the occurrence of grout voids was common in all bridges inspected and that a lack of grout facilitated rapid corrosion of the strands. The comment was also made in relation to the unsatisfactory state of non-destructive tests for detecting corrosion in prestressing strands.

**Masuhoro Bridge in Hokkaido, Japan (1957) [217]**

Whilst this example is not similar to the Sorell bridge, the methods used for investigating the correlation between steel condition and NDE techniques have been useful for the present investigation. In particular, the interpretation of half-cell data in relation to the condition of both reinforcing and prestressing steel has been significant.

**Post-tensioned structure in Berlin, Germany (1957/58) [194]**

As for Masuhoro Bridge, the bridge detailed in this paper is dissimilar to the Sorell bridge but provides some insight on the correlation between steel condition and NDE techniques. Photographic evidence of strand condition was also useful. The report showed that strand fractures induced by corrosion were mostly able to be detected using Magnetic Leakage Flux measurements.

**Hamanatua Stream Bridge near Gisborne, New Zealand (1966) [71]**

This technical report prepared for Land Transport New Zealand discussed several examples of structures built between 1960’s and early 1970’s which are predominantly prestressed. This report contains detailed information relating to materials properties and chloride/carbonation data of each bridge. Whilst structurally dissimilar, the information contained in these reports were reviewed due to the environmental similarities between these bridges and the Sorell bridge. The report documents inconsistencies between chloride levels and corrosion of prestressing steel, and evidence regarding the true condition of the Hamanatua Stream Bridge is inconclusive based on tests conducted.
2.8 Summary

Background information relating to the design and service life of the Sorell Causeway Bridge was presented in this Chapter. It was found that whilst the original intention of this bridge was for a long-lasting, maintenance-free structure, the opposite was true with corrosion-related issues causing the early decommission of the bridge in 2002. A number of field investigations were conducted on the bridge prior to its demolition, including chloride profiles, load testing and non-destructive techniques such as half-cell and resistivity surveys. These investigations showed variable results, and failed to provide DIER with objective information in which to more accurately predict the condition of the prestressing steel and thus the remaining serviceable life of the Sorell bridge. In response to these findings, Chapters 4 and 5 present more detailed results in relation to the pre-demolition field investigations for correlation with steel condition data found in Chapter 6.
Chapter 3

Corrosion of Steel in Reinforced and Prestressed Concrete

3.1 Introduction

The commercial use of reinforced concrete has been in place since it was patented in 1854 by William Wilkinson in the United Kingdom [168, 187], which recognises the complimentary actions of steel and concrete in terms of strength and ductility. The concept of prestressed concrete was pioneered by Eugene Freyssinet, developing the idea in the mid 1920’s and acquiring a patent in 1928 [58, 78]. As such, reinforced and prestressed concrete is one of the most commonly used and cost-effective building materials for use in various structural applications. However, whilst concrete affords some protection to the embedded steel, there are a growing number of structures that are being affected by reinforcement corrosion. Corrosion of reinforcement may cause:

- concrete section losses (due to spalling and cracking)
- reductions in flexural and shear strength (due to cross-sectional area losses in steel)
- loss of bond/development of load transfer (thus reducing the load capacity)
- reductions in the ductility of the structure, causing sudden and brittle failure

These defects have significant cost implications (as outlined in Chapter 1), and with aging structures this cost is set to increase. Therefore prior to discussing results relating to corrosion investigations obtained for the present work, an understanding of how and
3.2 Steel Passivation in Concrete

why corrosion occurs on reinforcement embedded in concrete is required. Section 3.2 reviews how steel is protected by the concrete, and Section 3.3 summarises the theory on the depassivation of the steel and subsequent corrosion initiation. Corrosion mechanisms are reviewed in Section 3.4 and rates of corrosion for these mechanisms are discussed in Section 3.5.

3.2 Steel Passivation in Concrete

The process of corrosion is defined as

"the destructive attack of a material through a reaction with its environment" [3]

or

"the deterioration of a metal by reaction with species in the environment to form chemical compounds" [131]

For steel, the iron constituents are unstable and will want to revert to its lowest energy state, which is iron oxide [87]. Thus with exposure to atmospheric oxygen and water, steel will lose section or "corrode". The rate of corrosion for atmospheric steel will depend on the ambient temperature and the amount of water and oxygen present in atmosphere, as well as other environmental influences, such as the presence of chloride ions in a marine environment [2].

When steel is embedded in concrete, it is generally agreed in the literature that a very thin passive oxide film forms over the rebar surfaces due to the highly alkaline conditions existing at the steel/concrete interface. These conditions form as a result of the dissociation of solid hydration products contained within the cement paste (principally Calcium Hydroxide, with lesser amounts of sodium and potassium hydroxides) in the pore water contained within the concrete. The resulting solution is highly alkaline with a pH of greater than 12.5, depending on the presence of sodium hydroxide and potassium hydroxide [209, 228, 240]. In this environment, the iron constituents in the steel are most stable as iron oxides and oxyhydroxides and thus the passive oxide layer is born. It is less than 10nm in thickness and a number of authors have classified this layer as being either
3.2 Steel Passivation in Concrete

magnetite ($Fe_3O_4$) or maghemite ($\gamma - Fe_2O_3$) [27, 63].

These concepts are reflective of research developed by Marcel Pourbaix [227], where the thermodynamic behaviour of metals in aqueous solutions was investigated [304]. From this research, Pourbaix developed a series of simplified thermodynamic equilibrium diagrams which determined the stability and solubility of chemical species depending on the system’s pH and electrochemical potential [2, 182]. These are otherwise known as Pourbaix diagrams. For the present research, Figure 3.1 shows a simplified Pourbaix diagram reflective of the basic corrosion process involving iron and water [21, 131, 166, 182]. Lines A and B represent the boundaries of water hydrolysis, where the regions above Line A represent oxygen evolution and regions below Line B represent hydrogen evolution. Between these two regions, water is said to be stable. The development and maintenance of the passive film will depend on the pH and electrochemical potential characteristics of the iron-water system. Figure 3.1 indicates that iron can be in a passive state when the pH ranges between 9 and 13.

![Figure 3.1: A simplified Pourbaix diagram for iron and water [21, 131, 166]](image-url)
3.3 Corrosion Initiation & Propagation

It is quite possible that the passive layer can be maintained over time and the steel remains protected from significant corrosion. However, this layer can dissipate or breakdown under certain conditions (known as depassivation). This will cause the steel to go from a passive state to an active state with respect to corrosion.

There are two mechanisms commonly attributed to depassivation. These involve the carbonation of concrete and the accumulation of chloride ions at the steel/concrete interface. Galvanic cell formation (contact of two dissimilar metals) and stray currents may also cause the destabilisation of the film however these are not considered relevant for the present study. Carbonation of concrete involves the neutralisation of the pH of the concrete adjacent to the steel. As can be seen in Figure 3.1, as the pH begins to drop the passive film will become unstable and decompose, and corrosion may subsequently initiate. The exact mechanism for chloride induced corrosion is still not fully understood [39, 131, 182], however it is usually presumed that the chlorides accumulate at the steel/concrete interface until a chloride threshold limit is reached at which point the passive film breaks down locally and corrosion initiates [21, 57, 209]. The threshold limit is specified in the literature varies considerably, however it is generally accepted that corrosion can initiate if the ratio of chloride ions to hydroxyl ions exceeds 0.6 [21, 139, 148]. These mechanisms are discussed in more detail in Chapter 5.

Once depassivation has occurred, corrosion of the steel can take place which is usually by aqueous corrosion in the presence of oxygen and water. An electrochemical cell is established, with the evolution of cathodic and anodic sites on the steel [2, 21, 209]. At the anodic site, the iron constituent from the steel will preferentially go into solution and release electrons where a neutral or alkaline condition exists, and subscribes to the reaction shown in Equation 3.1.

\[ Fe \rightarrow Fe^{+2} + 2e^- \] (3.1)
3.3 Corrosion Initiation & Propagation

These electrons will travel through the concrete to the positively charged cathodic site where oxygen and water exist and are subsequently being reduced (Equation 3.2).

\[ 2e^- + H_2O + \frac{1}{2}O_2 \rightarrow 2OH^- \]  

(3.2)

Where acidic conditions are dominant, the cathodic reaction involves the evolution of hydrogen gas, which is shown in Equation 3.3. This scenario is not representative of the concrete environment and will not be considered further in this discussion.

\[ 2e^- + 2H^+ \rightarrow H_2(g) \]  

(3.3)

These reactions form the basis of reinforcement corrosion in concrete and are shown graphically in Figure 3.2. The production of hydroxyl ions (OH\(^-\)) and ferrous ions (Fe\(^{2+}\)) will then combine to form rust, the volume and morphology of which depends on the availability of oxygen and water. Corrosion products are discussed in more detail in Chapter 6. With a continued supply of oxygen and water combined with the possibility of reduced pH or the presence of chloride ions, corrosion of the reinforcement can continue and lead to significant section losses, concrete cracking and spalling, and consequent load capacity reductions.

![Figure 3.2: Generalisation of the corrosion process [74]](image-url)
3.4 Corrosion Mechanisms

Depending on the environmental conditions, differing mechanisms of corrosion can take place on embedded steel. For reinforced and prestressed concrete, these include general corrosion, pitting or localised corrosion, or stress-corrosion cracking. A relatively new theory presented for consideration is the possible influence of microbiological activity. These are now briefly discussed.

3.4.1 General Corrosion

General corrosion occurs where corrosion takes place on a large scale, where the effects of corrosion are far-reaching and section losses are generally consistent. Typically this occurs where the surrounding concrete loses its alkalinity and the pH drops below 9. Oxygen must be readily available at the cathode for the reaction to progress. The production of voluminous corrosion products typifies this form of corrosion, and it is often accompanied by long cracks and zones of spalling in the adjacent concrete. The association of general corrosion with atmospheric steel corrosion is well documented, and it is predominantly observed on steel embedded within carbonated concrete [21, 25, 57, 131, 220, 283]. However, consistently high concentrations of chlorides along reinforcement may also produce profiles similar to those observed for general corrosion, making the effects of pitting less obvious [57, 283, 285].

3.4.2 Pitting Corrosion

Pitting corrosion is predominantly associated with the presence of chlorides [57, 79, 87, 132, 209]. Where general corrosion signifies a consistent and widespread activation of the steel, pitting will occur where the passive film has broken down locally. Pitting corrosion poses a more insidious threat to the serviceability of a reinforced or prestressed concrete structure, as localised section losses in the steel can lead to sudden failure and reduction in ductility. Carino cites studies that suggest that pitting may be up to eight times the average depth of corrosion [86]. The mechanism of pitting is not completely understood, however a basic summary of the principle ideas are shown in Figure 3.3 and discussed in the following paragraphs.
3.4 Corrosion Mechanisms

The basic cathodic and anodic reactions are the same as outlined by Equations 3.1 and 3.2. Where chloride ions are present, these migrate towards the positively charged anode and encourage further dissolution of iron ions in accordance with Equation 3.1. Once free, the iron can undergo hydrolysis to liberate hydrogen ions into the pit which subsequently reduces the local pH. An example of such a reaction is shown in Equation 3.4.

\[ \text{Fe}^{2+} + \text{H}_2\text{O} \rightarrow \text{FeOH}^+ + 2\text{H}^+ \]  

(3.4)

As the pH drops and in the presence of chloride ions, hydrochloric acid forms and stabilises the localised pH drop [131]. The pH within the pit is typically less than 5 at this stage [57]. Hydrogen gas may evolve under these acidic conditions, as shown in Equation 3.3, as may be Hydrogen Sulphide gas (H2S) as a result of Manganese Sulphide (MnS) inclusions within the steel [322].

Figure 3.3: Generalisation of pitting in mild steel [57, 65, 131, 182, 322]
3.4 Corrosion Mechanisms

Chlorides may also directly combine with the free iron to form iron chlorides (typically ferrous chloride) as they migrate away from the bottom of the pit (Equation 3.5) [182, 322]. These are thermodynamically stable at a low pH and will form a hydrated precipitate. As the concentration of the iron chlorides increase, these compounds will go back into solution, releasing the chloride ions to once again participate in the corrosion process [65, 182].

\[ 4H_2O + Fe^{2+} + 2Cl^- \rightarrow FeCl_2 \cdot 4H_2O \] (3.5)

For the pit to sustain growth the pH of the surrounding solution must become more acidic and an increase in chloride concentration must also occur (which involves the recycling of chloride ions) [283]. A porous crust may form over the top of the pit, which excludes the migration of oxygen into the pit forcing the dissolved iron to stay in solution; the formation of voluminous iron oxides is thus significantly reduced [57, 65]. The evolution of hydrogen may occasionally burst this crust. If these conditions are not met, the steel at the pit site may repassivate [79, 131].

3.4.3 Stress Corrosion Cracking & Hydrogen Embrittlement

Stress corrosion cracking (SCC) is a complex problem that is still not completely understood [125]. However, the basic definition of SCC is described as being

"the process in which the damage caused by stress and corrosion acting together greatly exceeds that produced when they act separately." [2]

In other words, cracking is induced in a steel section due to the combined actions of stress and corrosion. This phenomenon is more frequently observed in steels with increasing strength (for example high-tensile steel under stress), however SCC is also known to form in mild steel under certain environmental conditions [3, 105]. For prestressed concrete, strands may be susceptible to SCC at ambient temperature and low concentrations of chlorides and sulphides. Water vapour is also known to be influential in this process [105]. The physical manifestation of SCC starts as a microcrack in the steel, which is induced from small surface imperfections, pitting or just general rust spotting. It occurs without deformation and often without visible corrosion products [215]. The crack propagates over time, often in the form of intergranular or transgranular cracking, and its morphology is predominantly a function of the environmental conditions [3, 21, 57, 113]. Sudden
failure of the steel is characteristic of SCC; there is little necking and the fracture surface is brittle, however it should not be classified as a fatigue failure [246].

Hydrogen induced stress corrosion cracking (HI-SCC) is a form of SCC that is induced or accelerated by the presence of hydrogen gas. The failure due to HI-SCC is often called Hydrogen Embrittlement (HE), which is defined as

“the reduction in ductility due to the absorption of atomic hydrogen into the metal lattice.” [3, 57]

HE is sometimes classified separately to HI-SCC in the literature [3, 246], but as previously stated this area of research is still undergoing investigation. It appears that steels of yield strength less than 600MPa are unlikely to be affected by HE, and high strength steels (greater than 1000MPa) are significantly more at risk [57, 105]. There is no requirement for the steel to be in a stressed state for HE to occur [3, 21]. For prestressed concrete containing cold-drawn high tensile steel (as was the case for the Sorell Bridge), structures at risk of hydrogen gas evolution are those subjected to cathodic protection. This promotes the formation of hydrogen gas due to the inducement of highly negative potentials (as seen by the region below Line B of the Pourbaix diagram in Figure 3.1) [57, 105]. Hydrogen gas may also evolve when the steel is subjected to acidic corrosion (previously discussed in Section 3.4.2). The evolved hydrogen is absorbed into the crystalline structure of the steel and has a tendency to be attracted to areas of higher stress where the metal structure is dilated [105]. The hydrogen is then involved in the weakening of metal at these points, leading to embrittlement and eventual failure.

3.4.4 Microbiologically Influenced Corrosion

It is well documented in the literature that microbiological activity can be influential on corrosion process of steel in many industrial applications such as pipelines, pulp and paper production, chemical manufacture and food industries, as well as across steel infrastructure located in marine or polluted environment [59, 103, 142, 172, 190, 193, 247]. However there is very little information that documents the involvement of bacteria in corroding reinforced structures located in “normal” atmospheric conditions. Halstead and Woodworth [144] document findings of reinforced concrete members subjected to corrosion and state that “bacterial corrosion cannot be ruled out as an impossibility, although considered
to be unlikely" but no intentional testing is conducted to verify this statement. Little and Lee [172] cites a reinforced concrete dam in South America investigated by Mittleman and Danko in which sulphur-oxidising and sulphate-reducing bacteria were supposedly responsible for its deterioration. Little and Lee [172] showed that fungal bacteria were involved in the corrosion of post-tensioning strands, where biological action caused the decomposition of grease and the acidic by-product attacked the steel. Broomfield [65] provides a short but vague description of the possible involvement of bacterial corrosion in reinforced concrete. Melchers and Li [192] have documented corrosion observations on a 63 year old reinforced concrete balustrade which show symptoms similar to those affected by biological attack. Given these observations it would be reasonable to consider the possibility that microbiologically influenced corrosion (MIC) may have been involved in the deterioration of the Sorell Causeway Bridge.

There are several types of micro-organisms that can be involved in MIC. These include bacteria, fungi, algae and protozoans [247]. They can survive under a wide range of environmental conditions pertaining to pH, temperature and aeration. For the survival and colonisation of bacteria, they require [172]:

1. water
2. nutrients
3. electron acceptors

Electron acceptors include metals, especially iron. Nutrients include carbon, nitrogen, sulphur and phosphorous. The exact mechanisms of MIC are still under investigation, and there is even some speculation whether these micro-organisms are the cause of the corrosion or are simply present due to the corrosion activity (due to the presence of iron) [172]. The literature states that MIC initiates with the attachment of the micro-organisms to the steel surface [59, 142, 247]. A biofilm is established, protecting the organisms from detachment; this typically occurs 2 to 4 hours after attachment. The process of colonization then occurs, growing exponentially, with accumulation of dead cells and entrapped particles over the initial biofilm site. An electrochemical cell forms at this site due to oxygen depletion (leading to cathodic depolarization) and the development of chemical and physical gradients across the biofilm. A pit is initiated in accordance with the processes
3.4 Corrosion Mechanisms

described in Section 3.4.2, and the metal substrate begins to dissolve. Bio-corrosion is promoted due to the conditions inside the biofilm, which are acidic (due to the build-up of waste products) and depleted of oxygen.

As corrosion progresses, corrosion products can build up over the pit site. This biologically induced phenomenon is known as a tubercle, and it consists of three layers, containing the various oxidized states of the corrosion products. The innermost layer is mostly ferrous hydroxide ($Fe(OH)_2$), the middle layer is typically magnetite ($Fe_3O_4$) and the outer layer is ferric hydroxide ($Fe(OH)_3$). Green rusts are also known to exist within these layers [92, 126, 218]. These states depend on the amount of oxygen that is present across the tubercle; inner layers represent anoxic or anaerobic conditions, whilst the opposite is true for the outer layer (showing an aerobic condition). Corrosion products in relation to reinforcement corrosion are discussed in Chapter 6. A summary of the development of MIC is shown in Figure 3.4, which is based on mild steel exposed to flowing water. The formation of the tubercle shows the various stages of oxidation, as well as its position with regards to the biologically induced pit.

![Figure 3.4: Generalisation of pitting initiated due to microbiological activity [59, 173]](image-url)
Microorganisms are known to exist within two main zones of the tubercle, which are reliant on the availability of oxygen. Those that survive without oxygen are called anaerobic micro-organisms. In the literature, Sulphate-Reducing Bacteria (SRB) are the predominant organism discussed in relation to biocorrosion, as it was thought initially that this was primary cause of MIC, and it is thought to be one of the most damaging organisms [59]. SRB reduce sulphate ions to sulphide; the sulphides may form hydrogen sulphide ("rotten egg" gas), or soft, black iron sulphide when steel is present [59, 142, 173, 247, 306]. SRB can survive where the pH range is between 5 and 9.5, and are most active when temperatures are between 25 and 35°C [59, 247]. SRB are synonymous with pitting and produce distinctive pitting characteristics [145, 172, 306]. These characteristics include black, slime deposits adjacent to or over the pitted surface and a bright, metallic surface at the base of the pit [59, 142, 145, 247]. Shallow, concentric rings or steps are also indicative of SRB action [145, 234, 306].

Aerobic microorganisms are those that will colonise preferentially in the presence of oxygen. Typical aerobic organisms include sulphur oxidising bacteria, iron and manganese bacteria, slime-depositing bacteria and fungi. Iron/manganese related bacteria shall primarily be discussed, as it appears most frequently in the literature in relation to mild steel [59, 103, 247, 306].

Iron-oxidising bacteria (IOB), otherwise known as iron-depositing microorganisms, oxidise ferrous ions ($Fe^{2+}$) to ferric ions ($Fe^{3+}$). The former ion is soluble, whilst the latter is not. The ferric ions can combine with oxygen to form various forms of iron oxides; this is why IOB are normally associated with the formation of tubercles. However, the mechanisms and involvement of IOB in corrosion remains imperfectly understood at this time. Stalk-like Gallionella bacteria is the most commonly documented IOB in the literature [59, 103, 142] however other filamentous species include Sphaerotilus, Leptothrix and Crenothrix [59, 103, 142]. It is known that IOB can co-exist with SRB or can occur in isolation [263, 306].
3.5 The Rate of Steel Corrosion in Concrete

The rate at which steel corrodes within concrete depends on the properties of the surrounding concrete matrix and the reaction kinetics experienced by the cathodic and anodic zones [131, 283]. There are three mechanisms that will control the rate of corrosion: anodic control, cathodic control, and resistive control. Anodic control is regulated by the rate at which iron goes into solution after depassivation, whereas cathodic control is predominantly governed by the reduction of oxygen. Resistive control is based on the electrolytic resistance of the concrete between cathodic and anodic regions. Resistive control is likely to be the determining factor for the corrosion rate in general corrosion, whereas pitting is expected to be influenced by both resistive and anodic control.

Due to the variability of these factors and the surrounding environment, it is impossible to specify exact corrosion rates expected for various scenarios. However, estimates can be provided. Very high corrosion rates are those considered above 100 $\mu$m/year; less than 2 $\mu$m/year relates to negligible corrosion rates. Cross-sectional area losses are estimated to be 1 $\mu$m/year for passivated steel in concrete [131]. Bertolini et al [57] cites work done by Andrade et al [24] which summarises the range of corrosion rates to be expected under various conditions, and is reproduced in Figure 3.5. The greater the chloride contamination and relative humidity, the higher the rate of corrosion, with rates of between 100 and 1000 $\mu$m per year, although the percentage of chloride contamination is not specified. Note that the difference in rate between the aforementioned severe case is approximately 10 times the rate observed for the worst-case scenario for carbonated concrete. It does not appear that these rates take into consideration temperature, however it is anticipated that an increase in temperature would result in an increase in the corrosion rate.
Figure 3.5: Comparison of corrosion rates for various chloride and carbonation scenarios [57]

3.6 Summary

The mechanisms of reinforcement corrosion have been discussed in this chapter, with the introduction of another mechanism involving microorganisms. The following chapters will now discuss results relating to commonly used field investigative techniques for identifying areas at risk of corrosion.
Chapter 4

Non-Destructive Testing

4.1 Introduction

The origins of non-destructive testing (NDT) for evaluating the risk of corrosion to reinforced concrete structures are founded in the 1950’s. It was motivated by the increasing occurrence of spalling and rust staining of reinforced concrete decks which was attributed to the application of de-icing salts. Since that time, these techniques have evolved and developed into a wide variety of commercially available tests.

It is the purpose of the present investigation to evaluate NDT results conducted on the Sorell Causeway Bridge prior to its decommission; thus techniques such as linear polarisation resistance is not considered. All three test beams (Beams 17/4, 17/3 and 118) have undergone more detailed NDT, with focus predominantly on Bay 5 of each beam due to the presence of corrosion-related defects.

The chapter structure is as follows: Section 4.2 discusses the initial visual observations made on the beams prior to load testing. Section 4.3 reports on the cover readings measured on each test beam, especially the web shear ligatures in Bay 5. Sections 4.4 and 4.5 will then discuss results pertaining to the half-cell and concrete resistivity tests respectively. Information regarding the adopted nomenclature for the test beams and investigations is discussed in Appendix A. The relevant methodologies and background information for these tests are found in Appendix B. Detailed results are found in Appendix C.
4.2 Visual Inspection of Beams

4.2.1 Introduction

The visual inspection is considered to be one of the most crucial elements in the condition assessment of a structure. It is a relatively simple and uncomplicated procedure, which has a long history of use and is still the most common and widely used form of non-destructive evaluation today [225]. There are many definitions as to what a visual inspection is and what it comprises. However, all sources agree that it serves as a starting point of any condition assessment, providing an initial general impression of the condition of the structure [4, 47, 57]. The ACI Committee report states that “initial impressions can be very valuable [as they] often accurately characterize the nature of a problem” [4].

The aim of this section is to present the results of a detailed visual inspection conducted on Beams 17/4, 17/3 and 118, the majority of which are photographic records and crack mapping found in Appendix C. Comparisons shall then be made using these observations to the physical condition of the steel (see Chapter 6) in order to determine the degree of correlation.

4.2.2 Background Information on Visual Inspections

Visual inspections are an entirely non-destructive technique (i.e. they do not require the destruction or removal of small sections of the structure) and do not require any special equipment for the technique to be carried out. However, it is not a structural survey as it is based purely on what can be seen with the naked eye with no further analysis [66]. The goal of a visual inspection is stated as thus [4]:

“to provide initial information regarding the condition of the structure, the type and seriousness of the problems affecting it, the feasibility of performing the intended rehabilitation, and information on the need for a detailed investigation.”

Therefore, visual inspections provide a first impression of the problems associated with the structure and the extent of damage. It follows that the inspector will therefore need to look for and carefully examine obvious signs of deterioration on the structure [82]. Defects such as surface delaminations, spalling, warping, movement, cracking, deterioration
4.2 Visual Inspection of Beams

or any other irregularities are identified and detailed within this technique. Visual inspections can also provide valuable historical insight of a structure’s performance over time. Poston et al [225] states that condition assessments conducted over a period of time as part of an asset management program can verify the structural integrity of a structure. It also provides information regarding the progression of recorded defects, allowing the asset owner to assess the rate of the deterioration. Dimensional details and confirmation of as-built or repair information of the structure should be included in any visual inspection [4].

Limitations of this technique include:

- Only defects/deterioration that are visible on the surface of the structure are noted, therefore some defects may not be immediately identified
- Some areas of the structure may be inaccessible or difficult to reach without assistance (for example, a harness or a scissor-lift inspection vehicle)
- The inspector may not always have experience in conducting visual inspections and may lack knowledge of what comprises a significant structural defect
- Inspection reports may reflect the subjective opinion of the inspector, regardless of established classification systems

In any case, visual inspections remain an excellent starting point for any condition assessment.

4.2.3 Equipment & Methodology Adopted

Information relating to the equipment and methodology used for conducting the visual observations is reviewed in Appendix B. In summary, the methodology follows the recommendations outlined in “Guide to Concrete Repair and Protection” [150], “Monitoring of steel corrosion in concrete” [50] and “Guide for Evaluation of Concrete Structure Prior to Rehabilitation” [4]. Tasks conducted included the dimensional verification of beams, detailed photographic records of all beams and more specifically of defects, and the mapping of cracks observed in Bay 5 of Beams 17/3 and 118. Beam 17/4 had no significant cracking and did not require crack mapping.
4.2 Visual Inspection of Beams

4.2.4 Results of Initial Visual Observations

As previously stated in Chapter 2, Beams 17/4, 17/3 and 118 were chosen based on their varying visual condition ranging from good to extremely poor. The following section provides a summary of the visual observations made for each beam, which is assumed to provide a visual indicator of underlying defects. These visual observations shall be compared with other test results and the physical condition of the reinforcement, detailed in Chapter 7. Full visual observation records are found in Appendix C.

4.2.4.1 Summary of Visual Observations

With the exception of spalling on Beam 17/4 and instances of longitudinal web cracking on Beams 17/3 and 118, all beam surfaces were found to be in reasonable condition. All beams conformed to the drawing specifications. Demolition damage was evident, with the observation of concrete sectional area losses and exposed reinforcement along the upper flanges of all beams. The majority of the exposed reinforcement had brown surface rust however no substantial cross-sectional losses were observed. Evidence of previous investigations by Façade Technology and Taywood Engineering were noticeable in both the webs and base flange soffit of Bay 5 in Beam 17/3, with concrete discolouration and grout slurry splatter across the bay adjacent to the longitudinal web cracks (see Chapter 2).

Concrete surfaces were hard and generally smooth, including areas adjacent to longitudinal web cracks in Beam 17/3. Minor, isolated surface delaminations were found on both faces of Bays 2 & 3 of Beam 17/4, but did not appear to stem from corrosion-related defects. Small, scattered blow holes (less than 10mm in diameter) were observed on isolated faces of Beam 17/4. White “blotches” and chalk-like stains were observed along the lower half of Beam 118 on most faces, in particular those on Face D (Figure 4.1). Similar irregularities were observed on isolated sections from both Beam 17/4 and 17/3, but not to the extent observed on Beam 118.

A number of spalls and rust staining due to reinforcement corrosion was observed on Face C of Bays 4 and 5 from Beam 17/4, with the latter bay exhibiting the worst condition. The defects were predominantly confined to the shear ligatures closest to Diaphragms
4.2 Visual Inspection of Beams

Figure 4.1: White, “blotchy” surface on the base of Beam 118

(iv) and (v), with general corrosion evident the full web depth of these bars (see Figure 4.2). Spall depths did not exceed 20mm. Whilst some of these bars were exposed, it was difficult to identify the depth of section losses across these bars. Similar, yet smaller scale incidences of these defects were found on Face D of Beam 17/3, on shear ligatures closest to Diaphragm (v). Rust staining was evident along crack edges at regular spacings (approximately 200mm in places) which were assumed to be corrosion of the shear ligatures but this could not be confirmed visually.

Figure 4.2: Spalling over shear ligatures in Beam 17/4

A longitudinal crack and associated spalling and rust staining was observed along the lower flange edge of Face D of Bay 6 from Beam 118. Removal of the concrete delaminations revealed a section of longitudinal reinforcement undergoing active corrosion (Figure 4.3). Section losses were unable to be determined but they were thought to be significant.
Additional spalling and rust staining was found along the base edges of some diaphragms of all beams, in particular Diaphragm (iv) of Beams 17/3 and 118 (Figure 4.4). In some cases, corrosion was associated with poorly compacted concrete or severe cracking. The defects observed in these diaphragms were noted to be extensions of the longitudinal web cracking found in Bay 5. These and additional instances of cracking are discussed further in Section 4.2.4.2.
Superficial delaminations were observed over end-bay bearing plates in all beams (Figure 4.5). Cover was extremely low in these locations (typically less than 5mm). Corrosion spots were observed on some exposed bearing plates, but no significant section losses were evident.

Figure 4.5: Example of superficial spalling over bearing plates at beam ends

4.2.4.2 Observation of Longitudinal Web Cracking in Beams 17/3 and 118

A variety of different cracks were observed across all beams, however the predominant and most significant cracking was the longitudinal web cracking observed in Bay 5 of Beams 17/3 and 118 (Figure 4.6 and 4.7 respectively). The cracking appeared to follow the trajectory of the post-tensioning tendons, running along the webs and into the lower flange towards Diaphragm (iv). Smaller, secondary cracking running parallel to the lower crack further up the web was observed. Cracking extended diagonally through Diaphragm (iv).

Beam 17/3 exhibits the worst of the cracking (Figure 4.6a). Maximum crack widths of approximately 20mm were recorded on both faces; crack widths on Face D were very large almost the full length of the bay (greater than 10mm), with the appearance that the lower flange could detach from the beam (Figure 4.6b). Very little rust staining was evident at the mouth of the crack on both faces, with the exception of spalling over corroding shear
4.2 Visual Inspection of Beams

ligatures on Face D. Concrete adjacent to the cracks remains hard and predominantly defect free. Patch repairs and evidence of previous investigations (such as cores and crack gauges) are evident on both faces.

Bay 5 of Beam 118 shows symptoms similar to Beam 17/3, especially on Face C (Figure 4.7a). However, large amounts of spalling were associated with web cracking on Face D, with exposed concrete and large, deep voids opening up to what appeared to be the lower prestressing tendon (Figure 4.7a). The concrete on Face D (and adjacent to spall zones) had a “mottled” appearance with a chalk-like residue accumulating adjacent to observed spalling. Despite these observations, concrete appeared hard and smooth on both faces. Slightly more rust staining was evident, especially at centre-bay of Face C, but as for Beam 17/3 it is not significant. Crack widths were not as significant as those measured for Beam 17/3 (Figure 4.7b). Maximum widths of almost 10mm were recorded, however these were found at spall zones where concrete surfaces had opened significantly. Actual crack widths in the worst areas ranged between 3 and 5mm typically. Similar longitudinal cracking was observed across Bay 4 of both beams, which appeared to be an extension of cracking from Bay 5. These cracks were not as severe, with widths less than 1mm. These patterns can be viewed in Appendix C.

Figures 4.6b and 4.7b show additional longitudinal cracking observed along the soffit of the base flange in both Beams 17/3 and 118. These cracks stemmed from Diaphragm (iv) in both cases and ran for lengths of approximately 1m. Crack widths ranged between 0.2 to 7.5mm (Beam 118 exhibited the worst cracking in this instance), with the widest cracking observed 500-700mm from the diaphragm edge. Orange-brown rust staining was evident at isolated locations along the crack edges of Beam 17/3, however it was not significant or widespread. Figure 4.8 shows an example of the crack orientation and features found on the soffit of Bay 5 of Beam 118. Other soffit cracks were observed along the base of Bays 3 and 4 of Beam 17/3 and Bays 2 and 4 of Beam 118, of which a selection are shown in Figure 4.9. Crack widths typically did not exceed 3mm in these instances and were occasionally accompanied by rust staining. Crack lengths also varied, often ranging between 300 and 400mm.
4.2 Visual Inspection of Beams

(a) Photographic record of cracking

(b) Crack widths

Figure 4.6: Crack patterns & widths of longitudinal web cracks on Bay 5, Beam 17/3
4.2 Visual Inspection of Beams

(a) Photographic record of cracking

(b) Crack widths

Figure 4.7: Crack patterns & widths of longitudinal web cracks on Bay 5, Beam 118
4.2 Visual Inspection of Beams

Additionally, a number of smaller, random, longitudinal cracks were observed in various bays in Beams 17/3 and 118. These included Face D of Bay 4 from Beam 17/3 and Face D of Bays 2 and 4 from Beam 118, examples of which are shown in Figure 4.10. The cracks were predominantly confined to the side edges of the base flange, ran for approximately 300mm and rarely exceeded widths of 1mm. Some rust staining was evident. Note that
these cracks coincide with the location of cracks observed in the previous paragraph and shown in Figure 4.9.

![Image](image.png)

(a) Bay 4 of Beam 17/3  
(b) Bay 2 of Beam 118

Figure 4.10: Instances of cracking along base flange edges

### 4.2.4.3 Condition of the Transverse Tendons

Good viewing access was afforded to the transverse ducts located in the diaphragms. It was found that the majority of the transverse stressing strands were centrally located within each duct (most likely due to the absence of duct draping) and were completely surrounded by dense, hard grout in most cases. The condition of these cables was found to be excellent, with only minor surface rust evident at steel ends exposed to the atmosphere. In some cases, the mortar packing that was installed between adjacent beams during construction was still in place. Most were in relatively good condition with minimal voids and dense packing.

Whilst the majority of these tendons were in good condition, some were observed to have incomplete grouting. Subsequently, strand corrosion was evident at these locations. Table 4.1 provides a summary of the grout condition of the transverse tendons for each beam. The condition is expressed as a percentage loss of cross-sectional area of the tendon by inspection, with 100% being fully grouted. The number and distribution of tendons with incomplete grout is small, but every beam has at least one occurrence. Transverse grouting for Beam 118 was found to be in the best condition.

The worst case scenarios were observed on the base tendon of Diaphragm (ii), Beam 17/4 (Face C), and the top tendon of Diaphragm (iv), Beam 17/3 (Face C) (Figure 4.11). The
Table 4.1: Summary of grout condition in diaphragm transverse tendons for each beam

<table>
<thead>
<tr>
<th>Diaphragm No. &amp; Location</th>
<th>Beam 17/4 %</th>
<th>Beam 17/3 %</th>
<th>Beam 118 %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FC</td>
<td>FD</td>
<td>FC</td>
</tr>
<tr>
<td>(i)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>(ii) top</td>
<td>90</td>
<td>98</td>
<td>100</td>
</tr>
<tr>
<td>(ii) base</td>
<td>80</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>(iii) top</td>
<td>100</td>
<td>95</td>
<td>98</td>
</tr>
<tr>
<td>(iii) base</td>
<td>100</td>
<td>100</td>
<td>95</td>
</tr>
<tr>
<td>(iv) top</td>
<td>100</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>(iv) base</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>(v)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

former scenario had an obvious 10mm gap at the top of the tendon and corrosion has commenced on steel (Figure 4.11a). For Diaphragm (iv) of Beam 17/3, the grout in the tendon was not only insufficient, but had begun to deteriorate, exposing a greater number of strands to atmospheric conditions (Figure 4.11b). Corrosion had initiated on strands from Face D (Figure 4.11c).

These instances were not isolated, as several beams retrieved during the demolition of the bridge exhibited similar symptoms (Figure 4.11d). Based on these observations, it appears that DIER’s concerns regarding insufficient grouting can be confirmed. It stands to reason that if grouting was poor in the transverse tendons, there is a likelihood voids may also be present in the longitudinal tendons. This is discussed further in Chapter 6.

4.2.4.4 Summary of Visual Observations

In summarising the visual inspection, the three test beams were found to be in agreement with the design and construction drawings. Beam 17/4 was noted to be in the best condition, with minor spalls and shear ligature corrosion observed on Bays 4 and 5. Beam 17/3 was in the worst condition. Both Beam 17/3 and 118 had instances of longitudinal web cracking on both faces of Bays 4 and 5 (Bay 5 being the worst), with cracking from Beam 17/3 approaching consistent widths of 20mm. Very little rust staining is evident in these locations. Beam soffit cracking was observed adjacent to longitudinal web cracking.
4.2 Visual Inspection of Beams

on Beams 17/3 and 118 with minor rust staining. A number of voids were observed in diaphragm tendons, confirming fears of incomplete grouting. A review of steel and grouting conditions in the longitudinal tendons is presented in Chapter 6. Chapter 7 will review the degree of correlation between the steel condition and visual observations made in this chapter.

![Diaphragm (ii) of Beam 17/4](image1)
![Diaphragm (iv) of Beam 17/3 (Face C)](image2)
![Diaphragm (iv) of Beam 17/3 (Face D)](image3)
![Example of poor grouting](image4)

Figure 4.11: Examples of incomplete grouting of transverse tendons
4.3 Covermeter Survey

4.3.1 Introduction

Cover surveys have become commonplace in most reinforced concrete condition assessments, in which a electromagnetic covermeter is used to identify reinforcement patterns and provide estimates on steel depths. It is well documented throughout the literature that poor or low cover is likely to lead to earlier deterioration of reinforced or prestressed concrete structures due to corrosion [57, 66, 69, 73, 75, 87, 150, 299]. Therefore potential corrosion risks may be mitigated or addressed by identifying zones of low cover. The following sections provides some background information relevant to this method, the standards and methodology adopted for the present work, and the findings from cover surveys conducted across Beams 17/4, 17/3 and 118.

4.3.2 Overview of the Covermeter

Magnetic methods for use in structural engineering applications may extend back as far as 1905 as cited by Lauer [169], with mention of the employment of magnetic techniques to detect defects in magnetic materials (such as iron). The technique became commercially viable and more commonly accepted for use in condition assessments with the development of the “covermeter” by the Cement and Concrete Association in England in 1951 [75].

The method is based on the magnetic field phenomena, which involves ferromagnetic materials, substances that are strongly attracted to themselves and each other when magnetized [75, 169, 181, 288]. Although there are a number of aspects to this phenomena, only magnetic induction will be considered in relation to the covermeter. The basic principle involves the field effects of an electromagnet in response to steel reinforcement. To measure this response, an alternating current (supplied from a battery) is passed through the first coil of the electromagnet, and the response current induced in the second core is obtained in amplified form. The response from the second core will depend on the interference from any magnetic materials present in the vicinity of the electromagnet, such as reinforcement. When the electromagnet is passed over the steel, the induced magnetic field is distorted and will increase. The degree of distortion is a function of the diameter of the rebar and distance separating the rebar and the electromagnet, however this is
a non-linear relationship. Modern covermeters apply a calibration to account for this relationship, providing a more accurate measurement of the concrete cover.

### 4.3.3 Factors Influential on Results

Whilst the covermeter is an efficient tool for identifying reinforcement, there are a number of limitations that need to be considered prior to surveying reinforced concrete and interpreting results. These factors have been reviewed by Baker [54], Broomfield [66], Bungey and Millard [75], Lauer [169] and Manning [181], and the standard guides BS1881:204 [73], the Standards Australia publication HB-84 [150] and the NDT Guidebook [288]. Table 4.2 provides a list of these factors and the likely effects on the measured cover.

**Table 4.2: Summary of factors affecting covermeter readings**

<table>
<thead>
<tr>
<th>Factor</th>
<th>Causes</th>
<th>Effects on Reading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement Congestion</td>
<td>Large quantities of reinforcement; tie wire or bar chairs; splice bars</td>
<td>decreased cover</td>
</tr>
<tr>
<td>Bar Diameter</td>
<td>Small bar diameters (≤ 10mm)</td>
<td>increased cover</td>
</tr>
<tr>
<td></td>
<td>Large bar diameters (≥ 32mm)</td>
<td>decreased cover</td>
</tr>
<tr>
<td>Depth of Cover</td>
<td>Depths ≥ 100-300mm</td>
<td>varies; depends on bar diameter</td>
</tr>
<tr>
<td></td>
<td>Depths ≤ 20mm</td>
<td>varies; depends on bar diameter</td>
</tr>
<tr>
<td>Concrete Composition</td>
<td>Ferromagnetic compounds in concrete</td>
<td>decreased cover</td>
</tr>
<tr>
<td>Surface Condition</td>
<td>Irregular, rough surface</td>
<td>depends on surface variations</td>
</tr>
<tr>
<td>Corrosion Products</td>
<td>Corrosion products adjacent to steel</td>
<td>reduced cover / false location</td>
</tr>
<tr>
<td>Reinforcement Orientation</td>
<td>Skewed or misaligned reinforcement</td>
<td>varies</td>
</tr>
<tr>
<td>Temperature</td>
<td>Temperatures outside of covermeter operating range</td>
<td>gross errors</td>
</tr>
</tbody>
</table>
4.3 Covermeter Survey

4.3.4 Cover Requirements Based on Drawing Specifications

Prior to conducting the cover survey, the design drawings were consulted to determine reinforcement patterns and placements, as well as the cover specified. Design drawings 509-S46A, 47 and 48 were predominantly used, with some input from the design documentation [110]. Figure 4.12 summarises the design cover requirements for the beams and Figure 4.13 shows extracts from the design specifications from Drawings 509-S47 and 48.

Figure 4.12: Summary of cover requirements to shear ligatures and prestressing tendons based on drawing specifications (units in mm) [110]

4.3.5 Equipment & Methodology Adopted

The equipment used and the procedures adopted for measuring beam covers for experimental work is reviewed in Appendix B. In summary, a PROCEQ 4 covermeter was used in accordance with the British Standard BS1881:204 [73]. Cover to shear ligatures was determined from the webs and base flange soffit.
4.3 Covermeter Survey

(a) Reinforcement cage layout for both end bays and internal bays (Drawing 509-S48)

(b) Prestress and End Bay reinforcement layout for internal beam (Drawing 509-S47)

Figure 4.13: Reinforcement and prestressing layout for an internal beam based on drawing specifications [110]
4.3 Covermeter Survey

4.3.6 Results of Covermeter Survey

Tables 4.3a and 4.3a summarises the covers measured to the shear ligatures on the webs and the beam soffit respectively. More detailed results for each beam are found herein. In summary, Beams 17/3 and 118 were found to have an average cover lower than the design specification. Beam 17/4 had an average cover slightly higher than the specification, however individual results show that web covers measured on Face C were significantly less than those on Face D (refer to section 4.3.6.1). As a general rule, shear ligature covers decreased towards the base of the web.

Table 4.3: Summary of covermeter survey results

(a) Cover to shear ligatures across the webs

<table>
<thead>
<tr>
<th>Units in mm</th>
<th>B17/4</th>
<th>B17/3</th>
<th>B118</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>70</td>
<td>58</td>
<td>60</td>
</tr>
<tr>
<td>Minimum</td>
<td>5</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Average</td>
<td>30.6</td>
<td>23.4</td>
<td>25.4</td>
</tr>
<tr>
<td>Variance</td>
<td>103.9</td>
<td>50.8</td>
<td>45.3</td>
</tr>
<tr>
<td>Std. Dev'n</td>
<td>10.2</td>
<td>7.1</td>
<td>6.7</td>
</tr>
<tr>
<td># Readings</td>
<td>582</td>
<td>473</td>
<td>487</td>
</tr>
</tbody>
</table>

(b) Cover to shear ligatures along the beam soffit

<table>
<thead>
<tr>
<th>Units in mm</th>
<th>B17/4</th>
<th>B17/3</th>
<th>B118</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>30</td>
<td>-</td>
<td>27</td>
</tr>
<tr>
<td>Minimum</td>
<td>5</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td>Average</td>
<td>17.3</td>
<td>-</td>
<td>22.3</td>
</tr>
<tr>
<td>Variance</td>
<td>35.6</td>
<td>-</td>
<td>22.5</td>
</tr>
<tr>
<td>Std. Dev'n</td>
<td>6.0</td>
<td>-</td>
<td>4.7</td>
</tr>
<tr>
<td># Readings</td>
<td>72</td>
<td>-</td>
<td>16</td>
</tr>
</tbody>
</table>

Covers along the beam soffit were generally less than covers measured across the webs, averaging between 17 and 22mm. Excluding the end bays (where cover readings were influenced by reinforcement congestion and the steel bearing plates), minimum covers of 5mm were recorded for all beams in various locations. Many of these results corresponded to zones of spalling, rust stains or exposed reinforcement, including Face C of Bay 5, Beam 17/4. Covers measured on Bay 5 for Beams 17/3 and 118 were not exceedingly low, averaging approximately 24.5mm.

Variations of ±1mm were found between covers obtained using the covermeter and those physically measured, which for such small rebar is a unusually large error. Some agreement was found between the observed reinforcement placement and the drawing specifications. Shear ligature spacings on the whole were found to conform to the 205mm spacings (approximate)(Figure 4.13a). Some misplacement and rotation of reinforcement cages was observed, leading to the base longitudinal bars (and subsequently tendons) to be markedly off-centre (Figure 4.14). Cages were often observed to have shifted downwards,
4.3 Covermeter Survey

decreasing the amount of cover to the soffit of the beam. Several shear ligatures were also noted to be bowed or distorted (Figure 4.15). It is thought that these observations and other factors outlined in Table 4.2 (such as corrosion product build-up and surface irregularities) may have contributed to the errors observed.

As previously stated, reinforcement congestion and bar diameter settings on the covermeter prevented accurate cover readings to be obtained for the prestressing steel. However by inspection the tendon placement was found to be in accordance with the drawings. The exception to this statement was Beam 17/4. The following sections will provide more detail on results gathered for Beams 17/4, 17/3 and 118 respectively. Full covermeter results can be found in Appendix C.

4.3.6.1 Beam 17/4

Results are summarised statistically in Figure 4.16. Results for Face C consistently demonstrate a reduction in cover (Figure 4.16a), whilst those recorded for Face D were the opposite (Figure 4.16b).

Figure 4.17 shows a graphical representation of the cover data for both faces, where red areas distinguish zones of cover less than the design specification. It was noted that the majority of covers complied with the specifications on Face D compared to Face C (especially for Bays 2 and 5). Also note the trend of decreasing cover down the face of the webs. All bearing plates were clearly identifiable from the plots. Bay 5 registered the lowest average cover, with an average of approximately 16mm and a minimum of 6mm for
4.3 Covermeter Survey

Figure 4.16: Histogram of shear ligature web cover readings, Beam 17/4

Figure 4.17: Equicover contour plot for Faces C and D of Beam 17/4

Face C. The occurrence of low covers was denoted by zones of spalling and rust staining which coincided with the shear ligature placement (Figure 4.18).

Covers were visually confirmed for Bay 5 during demolition. Figure 4.19 shows a summary of covers measured in comparison to those measured from visual observations. There was general agreement between the two sets of results, with a maximum error of approximately 2mm. Note the shift of the reinforcement cage towards Face C, which had been governed by the location of the prestressing tendons. The cover to the prestressing tendons on Face C was less than the specification. This observation agrees with the cover patterns observed with the covermeter (Figure 4.17).
4.3 Covermeter Survey

Figure 4.18: Cover contour plots and visual overlay for Face C of Bay 5, Beam 17/4

Figure 4.19: Comparison of covers recorded for Bay 5 of Beam 17/4 from covermeter and physical measurements

4.3.6.2 Beam 17/3

Results have been summarised statistically in Figure 4.20. The majority of results did not comply with the design specifications, with Face C of Bay 6 being the only element with an average cover greater than 28mm (confirmed by the histograms).
4.3 Covermeter Survey

Figure 4.20: Histogram of shear ligature web cover readings, Beam 17/3

Figure 4.21: Equicover contour plot for Faces C and D of Beam 17/3

Figure 4.21 confirmed these findings, with only small pockets of cover in compliance. The minimum cover recorded was 5mm on Face D of Bays 3 and 5. Covers measured for both faces of Bay 5 (as shown in Figures 4.22a and 4.22b) were amongst those readings closer to compliance. Face D showed reduced cover towards the base of Bay 5, whilst the opposite was true for Face C. Zones of spalling and rust staining corresponded to very low covers recorded adjacent to Diaphragm (v) on Face D (Figure 4.22b). A similar rust stain was observed on Face C (Figure 4.22a), however covers in this region exceeded 30mm.
General agreement was again found between measured covers and those observed (Figure 4.23). Errors ranged between 1 and 3mm. The reinforcement cage was found to have rotated slightly, with lower covers observed towards the base of the beam on Face D. Covers around the top shear ligature remained consistent. Unlike Beam 17/4, the prestressing tendons were found to conform to the drawing specifications and complied with the cover specification.

Figure 4.22: Cover contour plots and visual overlay for Bay 5, Beam 17/3
4.3 Covermeter Survey

4.3.6.3 Beam 118

Results have been summarised statistically in Figure 4.24. Equicover contour plots for all faces of Beam 118 are shown in Figure 4.25. Similar observations and patterns were noted for Beam 118 as for Beam 17/3, with the majority of readings falling below the cover specification. However cover averages were more consistent between both faces and for each bay.

Figure 4.24: Histograph of shear ligature web cover readings, Beam 118
Minimum covers were not, on average, as low as those recorded for Beams 17/4 and 17/3, with the average minimum approximately 15mm. The lowest cover recorded was 7mm from Face C of Bay 6. As for Beam 17/3, increasing covers were observed up the web of Face C; the opposite was true for Face D (Figures 4.26a and 4.26b respectively). Evidence of spalling and rust staining did not appear to correlate to zones of low cover.

Less accuracy was observed between measured and observed covers, as shown in Figure 4.27, with a maximum error of 4mm recorded. The reinforcement cage had rotated in a similar fashion to that observed in Beam 17/3, and prestressing tendons were accurately placed in compliance with cover specifications.
4.3 Covermeter Survey

Figure 4.26: Equicover contour plots and visual overlay for Bay 5, Beam 118 (cont’d)

Figure 4.27: Comparison of covers recorded for Bay 5 of Beam 118 from covermeter and physical measurements

4.3.7 Summary of Covermeter Results

In conclusion, the majority of the covers measured were found to be less than that specified in the design documentation. Minimums as low as 5mm were recorded on all three beams. Some steel cage rotation and misplacement was evident from these results, especially from Beam 17/4. Observations of spalling generally coincided with zones of low cover.
4.4 Half-Cell Potential Survey

4.4.1 Introduction

Half-cell potential mapping is a technique commonly used in non-destructive structural evaluations. It provides an indication on the likelihood and extent of corrosion within a reinforced concrete structure, even if there are no visible corrosion signs on the concrete surface [49, 57, 63, 87, 177]. The technique was pioneered in the United States by R. Stratfull [261, 265–268] and is a relatively simple technique that requires little training and can be carried out economically and efficiently.

Ideally, this test is carried out in conjunction with several other tests in order to provide a holistic assessment of the structure in relation to the risks posed by corrosion [138, 302]. However, this test does not provide quantitative results regarding the amount of corrosion that has occurred [297]. The following sections provides an overview of the technique, factors that affect the results and summarises the equipment and methodology adopted for the present investigations. The results obtained for each test beam will then be discussed. Full details pertaining to nomenclature conventions are found in Appendix A. Details regarding project-specific methodology and equipment is covered in Appendix B, with full results contained in Appendix C.

4.4.2 Overview of Half-cell Potential Technique

The half-cell potential test is based on the premise that corrosion is an electrochemical process. This has previously been discussed in Chapter 3, but is briefly reviewed here. When an area of steel encased in concrete depassivates and corrosion initiates, an electrochemical cell is set up and current flows between the anodic and cathodic zones along the steel. These anodic and cathodic zones can be considered half cells in their own right, and electrons flow between these two points to create an electrical potential, or voltage, difference thus establishing an electric field [93, 96, 115, 195, 295]. This electric field can be detected on the surface of the concrete, and the potential difference is measured using a high impedance voltmeter.

A standard reference electrode is placed on the surface of the concrete in order to receive the electrical field information, as the potential differences between the anodic and
cathodic zones can vary due to the changes in electrolytic properties [295]. Standard electrodes are discussed in more detail in Appendix B. Any change that occurs in the potential in relation to the reference electrode can therefore be attributed to the activity of corrosion at the steel surface [96]. In order to register the corrosion activity between the anodic and cathodic sites, an electrical connection must be made between the reference electrode and the reinforcement. This requires the exposure of embedded steel that is free of rust scale [43, 93, 244, 295]. The half-cell potential method is shown diagrammatically in Figure B.2. This method was developed for and is predominantly used on reinforced concrete structures [86, 96, 115, 297], however there have been recent applications on prestressed and post-tensioned concrete structures [9, 71, 151, 214, 217].

![Diagram of half-cell potential method on reinforced concrete](image)

**Figure 4.28:** Idealisation of half-cell potential method on reinforced concrete [57, 87, 114]

### 4.4.3 Factors Influential on Results

A number of factors can influence half-cell potential results. These have been listed in Table 4.4 which have been summarised from the literature, namely Bertolini et al. [57], Chess & Gronvold [93], Figg & Marsden [122], Gu & Beaudoin [143] and Vassie [302], and
reference documents such as ASTM C876 [43], Austroads publication AP-127-97 [49], and the RILEM half-cell potential recommendation [244]. Other factors that can influence the half-cell potential result (but not mentioned in the table) include the presence of surface sealants or coatings, cementitious repair materials, epoxy-coated or galvanised reinforcement, the use of corrosion inhibitors at the time of mixing, accumulation of corrosion product in large cracks, magnetic rust layers (such as magnetite), and stray currents.

Table 4.4: Summary of factors affecting half-cell potential readings [43, 49, 57, 93, 122, 143, 244, 302]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Causes</th>
<th>Effects on Reading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cover depth</td>
<td>Large depths</td>
<td>Difficulty in identifying corrosion zones</td>
</tr>
<tr>
<td>Resistivity</td>
<td>Highly resistive surface layers</td>
<td>More positive shift in potentials</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Difficulty in identifying corrosion zones</td>
</tr>
<tr>
<td>Oxygen concentration</td>
<td>Oxygen-depleted environment</td>
<td>More negative shift in potentials</td>
</tr>
<tr>
<td>Chloride concentration</td>
<td>Presence of chloride additives</td>
<td>More positive shift in potentials</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flattening of potential gradients</td>
</tr>
<tr>
<td>Carbonation</td>
<td>Highly resistive surface layers</td>
<td>Highly positive potentials</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Introduction of junction potentials</td>
</tr>
<tr>
<td>Temperature</td>
<td>High/low temperatures change in relative humidity</td>
<td>Increase/decrease in potentials</td>
</tr>
</tbody>
</table>

4.4.4 Equipment & Methodology Adopted

A review of the methodology and equipment adopted for the present investigation is found in Appendix B. This includes a review of standard reference electrodes, sampling locations and interpretive guides to assist in identifying zones of corrosion (in accordance with recommendations from ASTM C876 [43], Chess & Gronvold [93], Figg & Marsden [122], the RILEM half-cell potential recommendation [244] and Vassie [302]).

In summary, half-cell potentials were obtained from most beam surfaces using the Copper-Copper Sulphate standard reference electrode (CSE). Electrical connections to the steel were varied to determine the half-cell patterns for both conventional reinforcement and prestressing strands. Absolute half-cell potential results were reviewed for likely risks of
4.4 Half-Cell Potential Survey

corrosion using the limits published in ASTM C876 [43], which are shown in Table 4.5. These empirical limits were initially proposed by Stratfull [265, 268] and further developed by Van Daveer [295] to reflect a probabilistic regime for corrosion. These limits are often referred to as the “Van Daveer criteria” [296, 317, 319]. In recent times, an additional limit has been included in this criteria, which provides an upper limit where evidence of corrosion is likely to be visible from the concrete surface [66, 259, 288].

Table 4.5: Half-cell potential limits recommended by ASTM C876 [43]

<table>
<thead>
<tr>
<th>Half-cell Potential Range, $E_0$ (CSE)</th>
<th>Probability of Corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_0 \geq -200$ mV</td>
<td>90% chance steel in that area is not corroding</td>
</tr>
<tr>
<td>$-200$ mV $\geq E_0 \geq -350$ mV</td>
<td>Corrosion activity is uncertain</td>
</tr>
<tr>
<td>$E_0 \leq -350$ mV</td>
<td>90% chance steel in that area is corroding</td>
</tr>
<tr>
<td>$E_0 \leq -500$ mV</td>
<td>Corrosion is likely to be visible on concrete surface [66, 259, 288]</td>
</tr>
</tbody>
</table>

4.4.5 Results of Half-cell Potential Surveys

This section provides an overview of the half-cell potential results obtained for each test beam. Detailed results are found in Appendix C. In general, half-cell potentials recorded for Beam 17/4 were more positive than Beams 118 and 17/3. Potentials for Beam 118 were, on average, more negative than Beam 17/3, but had a narrower potential range. There was a general trend for increasingly negative potentials towards the base of the beams and towards zones showing distress (such as the longitudinal cracks observed on Bay 5 of Beams 17/3 and 118). The minimum potential recorded was -804mV on the beam soffit of Bay 5, Beam 17/3. Some agreement was noted between the visual condition of the beams and the half-cell potential values as recommended by ASTM C876 [43]. A number of potentials on Beams 17/3 and 17/4 were recorded above zero (Beam 17/4 in particular) despite electrical connections being reconnected and surfaces re-wetted to reduced the resistivity (see Appendix B). Where this had occurred, the absolute value was disregarded, interpreting the likely risk of corrosion via potential gradients instead. The steel bearing plate on Bays 1 and 6 were easily identifiable from potential results.
4.4 Half-Cell Potential Survey

The following sections discuss the half-cell potential results with respect to corrosion of conventional reinforcement and prestressing strands, comparison of results with a variation in steel connection points and a preliminary assessment of results against defects observed. Detailed discussions relating to these results are found in Chapter 7, which incorporates the condition findings of the steel in Chapter 6.

4.4.5.1 Conventional Reinforcement

In relation to the conventional reinforcement, the statistical distribution of the results in relation to the ASTM C876 limits [43] are shown in Table 4.6. The majority of half-cell potentials measured for all beams were found to be more positive than -200mV, indicating that there is a low risk of corrosion according to the ASTM C876 limits. Beam 17/4 yielded the greatest proportion of more positive potentials. Potentials ranging between -200 and -350mV comprised the next greatest proportion of results for all beams. Beams 17/3 and 118 exhibited increasing numbers of potentials more negative than -350mV (in that order), with approximately 1% of results more negative than -500mV indicating that evidence of corrosion is most likely visible on the concrete surface. The majority of more positive potentials for all beams were obtained for Bays 1 to 3. Increasingly negative potentials were typically recorded for Bays 4 to 6, with Bay 5 statistically yielding the most negative results. Individual half-cell potential surveys for each beam in relation to the conventional reinforcement will now be discussed in the following sections, with particular focus on Bay 5 where the majority of visual defects were located.

Table 4.6: Distribution of half-cell potential results measured for conventional reinforcement

<table>
<thead>
<tr>
<th>Half-cell Potential Range, Eo (CSE)</th>
<th>Beam 17/4</th>
<th>Beam 17/3</th>
<th>Beam 118</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eo ≥ -200mV</td>
<td>77%</td>
<td>69%</td>
<td>41%</td>
</tr>
<tr>
<td>-200mV ≥ Eo ≥ -350mV</td>
<td>23%</td>
<td>19.5%</td>
<td>41%</td>
</tr>
<tr>
<td>-350mV ≥ Eo ≥ -500mV</td>
<td>0%</td>
<td>10.5%</td>
<td>16.5%</td>
</tr>
<tr>
<td>Eo ≤ -500mV</td>
<td>0%</td>
<td>1%</td>
<td>1.5%</td>
</tr>
</tbody>
</table>
4.4 Half-Cell Potential Survey

**Beam 17/4**

All vertical faces were measured for half-cell potentials prior to load testing, with two steel connections during the test. Potentials for the soffit of the base flange were obtained after load testing.

Results show that the majority of the potentials ranged between 0 and -300mV, as shown by the frequency histogram in Figure 4.29. In fact, over 80% of results lay within this range. The distribution of potentials between Face C and D were very similar, however just over 40% of potentials lay between -100 and -200mV for Face C whereas 35% of results for Face D were between 0 and -100mV. Potential distributions for the flange soffit revealed two distinct peaks, where 38% of results lay between 0 and -100mV and 46% of results were between -200 and -300mV. This shows a negative shift in potentials towards the base of the beam.

To investigate these results further, an equipotential plot of the entire beam has been constructed as shown in Figure 4.30, which includes results for both Faces C and D and the base flange soffit. Full results are not available for Bays 5 and 6 due to access issues. The trend for increasingly negative potentials towards the base of the beam was confirmed.
by Figure 4.30. Highly negative potentials existed on both faces of Bay 5, with Face C registering the absolute minimum potential recorded for Beam 17/4 of -340mV and identifying discrete minima ranging from -250 to -300mV (Figure 4.31). Note that some of the minima coincided with instances of spalling and corrosion of the shear ligatures but they did not always comply with the ASTM C876 limits. Minima were also observed along the base flange soffits for Bays 3, 4 and 5, however this did not translate to corrosion-related defects on the concrete surface. Bays 1 and 6 show localised corrosion zones at the base of the beam, which was attributable to the bearing plates at these locations.

Figure 4.30: Equipotential plot for Beam 17/4 (mV, CSE);
Electrical connection to conventional reinforcement

Figure 4.31: Equipotential plot for Face C of Bay 5, Beam 17/4 (mV, CSE);
Electrical connection to conventional reinforcement
Steeper equipotential gradients existed on the webs on several bays. Most webs had gradients less than 0.5mV/mm, however several areas exceeded this level (refer to the closely spaced equipotential lines on Bay 4 in Figure 4.30). Some examples include Face C of Bay 2 (Point A in Figure 4.30), which had a maximum vertical potential range between -50 and -200mV over 250mm, leading to a gradient of 0.6mV/mm. Face D of Bay 4 had a similar maximum vertical range between 0 and -200mV over 250mm, giving a gradient of 0.8mV/mm. The webs of Face C of Bay 4, Face D of Bay 5 and Face C of Bay 1 all exhibited maximum vertical gradients of 1mV/mm in several locations.

Less negative potentials were observed on both faces of Bay 3 and Face D of Bay 2, shown as blue regions marked by Point B in Figure 4.30. Highly positive results were observed on the upper web of Face C, Bay 3 with results averaging between 0 and +150mV. Positive readings may indicate that the concrete was very dry, that poor connectivity to the steel existed, there was an absence of chlorides, or that stray currents were present. The latter two options were ruled out as the beam contained calcium chloride (see Chapter 5) and the beams were tested in isolation to any external electrical currents. Steel connections were re-checked and the surface was also re-wetted in these areas. However, the concrete was found to be carbonated at some locations (see Chapter 5) which may have increased the surface resistivity and surface hardness, reducing the ability of the concrete to absorb moisture and thus influencing the results.

A variation in the steel connection point may also have influenced the results. This is demonstrated by the sharp change in potentials along the soffit of the base flange in Bay 2 (Point C in Figure 4.30). The distinct differences observed between the highly negative potentials on the soffit of Bay 3 and the highly positive potentials on the webs would appear to support this theory. Alternatively, steel electrical conductivity in this section may have been non-existent, or simply that the steel in this location was extremely passive.

**Beam 17/3**

The distribution of half-cell potentials obtained for the conventional reinforcement is shown in Figure 4.32. Potentials spanned over a wide range, from results greater than zero to a minimum of -625mV. Overall, approximately 50% of potentials lie between 0
and -200mV, and fewer than 25% of potentials lie between -200 and -400mV. From Figure 4.32, Faces C and D had very similar potential profiles, with a distinctive peak observed at -150mV. Face D had slightly more potentials ranging between -200 to -400mV, 24% in comparison to 16% for Face C. Potentials measured along the soffits of the beam showed a similar profile to Faces C and D. However these results were slightly more negative, with 30% and 25% of potentials ranging between -100 to -200mV and -300 to -400mV respectively. As for Beam 17/4, these results show that potentials were more negative towards the base of the beam.

The equipotential map for conventional reinforcement for the entirety of Beam 17/3 confirmed this trend in places (Figure 4.33). Very negative potentials existed on Bays 4 and 5, and it was expected that the potentials would be even more negative on zones from Bay 5 that were lost during load testing. Also noted on these bays were the steep gradients of the equipotential contours (Point A in Figure 4.33). In some cases vertical gradients up to 2mV/mm were consistently achieved. Several instances of local minima were observed across most web and soffit faces. The steel base plates at either end of the beam were also easily identifiable via the equipotential map.
Positive potentials were identified on Face C of Bay 2 and both faces of Bay 3 (Points B and C respectively on Figure 4.33). It is thought that there is either a lack of electrical connectivity of reinforcement for these bays or that, simply put, corrosion did not exist at these locations. For Face C of Bay 2, potentials reached a maximum of +170 mV. A series of regularly spaced maxima with positive potentials were located towards the top of the bay, which appear to follow the spacing of the shear ligatures.

**Beam 118**

The majority of potentials for Beam 118 relating to the conventional reinforcement were retrieved prior to load testing, as for Beams 17/4 and 17/3, with only one steel connection made at the top of Bay 4. Again, Bay 5 and the beam soffit were surveyed after this event, with steel connections varying between beam segments.

The half-cell potential frequency histogram is shown in Figure 4.34. Potentials ranged between 0 and -600 mV, with approximately 65% of results falling between -100 and -300 mV. Peaks at -100 mV and -200 mV were registered. All surfaces along the beam soffit and both Face C and D showed similar profiles and potential ranges, indicating that the probability of corrosion may not only be confined towards the base of the beam, as was found in Beams 17/4 and 17/3.

The corresponding equipotential map for Beam 118 is shown in Figure 4.35. More negative potentials were observed on Bays 2, 4 and 5. The minimum recorded for the entire
4.4 Half-Cell Potential Survey

beam in relation to the conventional reinforcement was -552mV, recorded on Face D of the base flange in Bay 2. The next highest minimum was recorded on the Face C web of Bay 4, registering -548mV, whereas in contrast the minimum recorded for Bay 5 was -502mV on the Face D web. Both the webs and soffit of Bay 3 showed the most positive potentials, reaching maximums of +50mV. In contrast to Beams 17/4 and 17/3, the absolute potentials measured along the beam soffit were relatively benign with the majority of potentials more positive than than -350mV. In reviewing the equipotential gradients, steep gradients between 1.0 and 1.5mV/mm were measured on the webs of Bays 2, 4 and 5 and the soffit of Bay 2.

Figure 4.34: Frequency histogram for half-cell potentials for Beam 118;
Electrical connection to conventional reinforcement

Figure 4.35: Equipotential plot for Beam 118 (mV, CSE);
Electrical connection to conventional reinforcement
4.4 Half-Cell Potential Survey

4.4.5.2 Prestressing Strands

Half-cell potential surveys obtained when connected to the prestressing steel was conducted primarily on Bay 5 of Beams 17/3 and 118 due to the presence of significant longitudinal web cracking that appeared to follow the trajectory of the prestressing strands. The surveys included web surfaces on both Face C and D and along the beam soffit. Electrical connection points were also varied, obtaining half-cell potentials for both upper and lower tendons. Prestressing steel potential surveys on Beam 17/4 were also conducted but on a significantly reduced scale, only taking measurements at various locations along the beam soffit.

A summary of the statistical distribution of these results in relation to the ASTM C876 limits [43] are shown in Table 4.7. The majority of results for Beam 17/4 and 17/3 were more positive than -200mV, as observed for the conventional reinforcement electrical connection (Table 4.7). In contrast, the majority of results obtained for Beam 118 were found between -200 and -350mV, indicating that corrosion activity is uncertain when referring to the ASTM C876 limits. Beam 118 showed a greater proportion of prestressing strands that showed potentials indicating that corrosion was highly likely to be occurring in relation to Beam 17/3, however the latter beam had a greater proportion of strands more negative than -500mV. Half-cell potential surveys for each beam in relation to the prestressing steel are now discussed.

Table 4.7: Distribution of half-cell potential results measured for the prestressing strands

<table>
<thead>
<tr>
<th>Half-cell Potential Range, $E_o$ (CSE)</th>
<th>Distribution of Results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Beam 17/4</td>
</tr>
<tr>
<td>$E_o \geq -200 \text{mV}$</td>
<td>83%</td>
</tr>
<tr>
<td>$-200 \text{mV} \geq E_o \geq -350 \text{mV}$</td>
<td>17%</td>
</tr>
<tr>
<td>$-350 \text{mV} \geq E_o \geq -500 \text{mV}$</td>
<td>0%</td>
</tr>
<tr>
<td>$E_o \leq -500 \text{mV}$</td>
<td>0%</td>
</tr>
</tbody>
</table>
4.4 Half-Cell Potential Survey

**Beam 17/4**

As previously discussed, minimal half-cell potential results were obtained for Beam 17/4 when connected to the prestressing strands, and were obtained from the beam soffits of select sections. The electrical connection was established via strands from the base tendon. The frequency histogram of these results is shown in Figure 4.36. Potentials fall between a narrow range of 0 and -350mV, with over 50% of results falling between -100 and -200mV. The minimum absolute potential recorded for prestressing strands from Beam 17/4 of -305mV was recorded at the centre of the beam soffit from Bay 2.

![Frequency histogram for half-cell potentials for Beam 17/4; Electrical connection to prestressing strands](image)

**Beam 17/3**

Figure 4.37 shows the frequency distribution of the potentials measured when connected to the prestressing strands for various scenarios. The large majority of potentials fell between 0 and -400mV (approximately 85% of total results), with a smaller peak observed between -600 and -850mV. Only a small number of positive potentials were recorded, with approximately 3% of results more positive than 0mV. Looking specifically at potentials
taken from both webs of Bay 5, Face C and Face D results had similar profiles. These results combine the potentials of strands from both the upper and lower tendons. Potentials ranged between 0 and -500mV and the majority of results lay between -100 and -300mV. Of these results, 28% fell between -300 and -400mV for Face D in comparison to 16% from Face C.

Reflecting on potential data of strands from the upper and lower tendons, the profiles were similar, ranging between 0 and -500mV. Potential distributions were almost identical, with the only exception being that 24% of strands from the upper tendon exhibited potentials between -300 and -400mV in comparison to 20% for strands from the lower tendon. The minimum potential recorded was -447mV at the base of Face D when connected to strands from the lower tendon. This data represents potentials taken from the webs of both faces on Bay 5 and excluded potentials measured along the soffit of the beam.

In comparison, potentials obtained along the beam soffit exclusively connected to the lower tendon reveal that the potentials were wide ranging between 0 to -850mV. There were two predominant peaks; the first peak has 71% of results fell between 0 and -300mV
and the second peak had 21% of results between -500 and -850mV, which related directly to the secondary peak initially observed for the overall results. The minimum potential recorded on the soffit was -804mV in Bay 5, which is substantially different to the minimum potential of -447mV recorded measured along the webs.

The equipotential maps for the upper and lower tendon from Bay 5 are shown in Figure 4.38. Similar patterns existed on both faces for either tendon, including instances of localised minima. For Face C, the lower tendon map had a slightly greater area of negative potentials than the upper tendon. For Face D, the opposite was true with more positive zones observed on the lower tendon map towards Diaphragm (iv). However, patterns and gradients for this case were slightly more pronounced than the upper tendon. Maximum gradients ranged between 1.0 and 1.5mV/mm.

**Beam 118**

As for Beam 17/3, additional steel connections were made to prestressing steel from the upper and lower tendons of Bay 5 in order to observe the half-cell potential patterns. Potentials corresponding to the prestressing steel were not obtained along the beam soffit, therefore all information gathered relates to both faces of the beam webs.

As seen in Figure 4.39, the potentials ranged between -100 and -550mV, with several peaks occurring at -400mV, -325mV, -275mV and -225mV. For all combinations, over 30% of potentials fell between -300 and -400mV. The potentials for Face C and D showed similar profiles, with the exception that almost half of the potentials recorded for Face D lay between -300 and -400mV, while potentials for Face C remained evenly distributed between -100 and -500mV. Potentials recorded of strands from the lower tendon were typically more negative than strands from the upper tendon, with almost 30% of results lying between -400 and -500mV in comparison to approximately 10% respectively. The minimum potential recorded was -480mV along the base of the Face C web when connected to strands from the lower tendon.
4.4 Half-Cell Potential Survey

Figure 4.38: Half-cell equipotential plot for Bay 5 of Beam 17/3 (mV, CSE); Electrical connection to prestressing strands
Figures 4.40a and 4.40b show the representative half-cell equipotential plots for the upper and lower tendon connections for Face C and D respectively. The plots were relatively similar between the two tendons, with the lower tendon typically exhibiting more severe patterns for both faces towards Diaphragm (iv) (in particular Face D in Figure 4.40b). Gradients along the tendons for all cases averaged between 0.5 and 1.0mV/mm. Steep gradients of approximately 2.0mV/mm were noted from several local minima observed over the upper and lower tendons in Figure 4.40b.
4.4 Half-Cell Potential Survey

Figure 4.40: Half-cell equipotential plot for Bay 5 of Beam 118 (mV, CSE); Electrical connection to prestressing strands
4.4.6 Half-cell Potential Plots in Relation to the Variation of Steel Connection

As an additional investigation, half-cell potentials and equipotential plots were compared in relation to variations in steel connection. Based on theoretical assumptions, if the reinforcement cage is electrically connected it should stand to reason that the equipotential plots for the same face (but different steel connection point) should be almost identical. It is also anticipated that different equipotential plots should occur between the conventional reinforcement and prestressing strands.

Figures 4.41 and 4.42 show the comparison between equipotential plots for Beams 17/3 and 118 respectively with varying steel connection points. Plots from both beams exhibited similar patterns in relation to the electrical connection. Indeed, similarities were also noted between faces. In terms of absolute values, slightly more negative potentials were recorded for conventional reinforcement, with maximums approaching -500mV.

Similar comparisons were made on equipotential plots along the beam soffits for Beams 17/3 and 118. Figure 4.43 shows an example of this comparison. Here the differences between steel connection points were more marked. For Bays 3, 4 & 6, potentials were more negative for the conventional reinforcement in comparison to the prestressing strands. However for Bays 2 & 5 the opposite is true with potentials recorded for the prestressing strands at least 100mV more negative than the conventional reinforcement. Based on these observations and the similarities of plots as seen in Figures 4.41 and 4.42, questions are raised regarding the influences from geometric boundary effects and “shadowing” of corroding reinforcement by near-by bars. These issues are discussed further in Chapter 7 in relation to the physical condition of the steel (Chapter 6).
4.4 Half-Cell Potential Survey

(a) Equipotential plot across Face C of Beam 17/3 with the steel connected to
   (a) FC rebar; (b) FD rebar; and (c) lower tendon strands

(b) Equipotential plot across Face D of Beam 17/3 with the steel connected to
   (a) FC rebar; (b) FD rebar; and (c) lower tendon strands

Figure 4.41: Equipotential plots with varying electrical connections for Beam 17/3
   with respect to varying steel connection points (mV, CSE)
4.4 Half-Cell Potential Survey

(a) Equipotential plot across Face C of Beam 118 with the steel connected to
   (a) FC rebar; (b) FD rebar; and (c) lower tendon strands

(b) Equipotential plot across Face D of Beam 118 with the steel connected to
   (a) FC rebar; (b) FD rebar; and (c) lower tendon strands

Figure 4.42: Equipotential plots with varying electrical connections for Beam 118
   with respect to varying steel connection points (mV, CSE)
4.4 Half-Cell Potential Survey

4.4.7 Preliminary Comparison of Half-cell Potential Results and Visual Observations

Although physical defects cannot be taken as definitive evidence of corrosion, half-cell potentials were reviewed in light of some of the more significant areas of cracking and spalling across all three beams. Preliminary observations have already been made for Beam 17/4, however it is the objective of this section to highlight specific issues pertaining to potentials obtained for Bay 5 of Beams 17/3 and 118 in relation to the presence of defects.

Figure 4.44 shows the surface of both faces of Bay 5 from Beam 17/3 with an overlay of the equipotential plot relative to the conventional reinforcement. A -300mV minima was approximately centred over a rust stain on Face C (Point A of Figure 4.44a), with a corresponding minima of -400mV on Face D (Point A of Figure 4.44b). Alternatively, a maxima of -150mV was centred over a spalling ligature on Face D (Point B of 4.44b). Note that this lies outside of the recommended ASTM C876 limits [43] for a high likelihood of corrosion.
4.4 Half-Cell Potential Survey

Figure 4.44: Comparison of equipotential plot and visual condition of Bay 5, Beam 17/3

Figure 4.45 shows a similar equipotential overlay for Bay 5 from Beam 118. Rust staining was evident at Point A on Face C (Figure 4.45a), which lay within a potential range of between -350 and -400mV, appearing to conform to the ASTM C876 limits [43]. However at Point B, potentials of -500mV existed but there were no external signs of corrosion (as per Broomfield’s recommendation [66]). Similarly, steep gradients existed at Point A on Face D (Figure 4.45a) with no rust staining. However, rust stains shown by Point B lay within potential zones of between -300 and -400mV (but without steep gradients). These results lay within the ASTM C876 limits [43].

With respect to longitudinal cracking and the identification of corrosion, there were no apparent trends. However, the observation of localised minima, a wider range of potential gradients, and lower potentials may indicate such zones (see Figure 4.46). Note the minima of -400mV at Point A on both Figure 4.46a and Figure 4.46b. Note also the steep potential gradients overlaying the corroding shear ligature on Face D (Point B on Figure 4.46b). Such trends will be discussed in further detail in Chapter 7 in relation to the physical steel condition (Chapter 6).
4.4 Half-Cell Potential Survey

Figure 4.45: Comparison of equipotential plot and visual condition of Bay 5, Beam 118

Figure 4.46: Comparison of equipotential plot and visual condition in relation to longitudinal cracking
4.4.8 Summary of Half-cell Potential Surveys

In concluding this section, the half-cell potential test overall appears to identify zones where corrosion was more likely to be occurring upon observing the equipotential plots (for example, Bay 5 had consistently more negative potentials than any other bay for each beam). On average, Beam 17/4 exhibited the most positive potentials, and Beam 118 the most negative. The minimum potential of -804mV was recorded on the soffit of the base flange, Beam 17/3. Similar results and equipotential patterns were observed between varying steel electrical connections and opposing faces. The interpretations made for the half-cell potentials results and equipotential plots in relation to the ASTM C876 limits [43] and the physical steel condition are discussed in Chapter 7.

4.5 Resistivity Survey

Measuring the resistivity of reinforced concrete has been in use since the 1950’s and provides qualitative information on the likely corrosion rate of the steel. It is a simple, efficient and cheap method used to assess structures on-site, but it is not as popular or widely used as the half-cell potential test [75, 288]. The resistivity test is typically carried out in conjunction with other non-destructive tests, especially the half-cell potential test and linear polarisation resistance technique.

The following sections provide a brief review of this test, including associated principles and factors affecting the obtained results, and present results obtained for each beam. Nomenclature conventions, adopted equipment and methodology, and detailed results are found in Appendix A, B, and C respectively.

4.5.1 Overview of the Resistivity Technique

As has been discussed previously in Chapter 4, the corrosion process depends on the development of potential differences between cathodic and anodic zones along the steel, which in turn generates a current flow pattern. This has been shown to relate directly to the rate of corrosion [54, 121]. For this process to occur, this requires concrete encasing the steel to exhibit electrical properties. Indeed, this is the case and concrete is often referred to as an electrolyte due to its ability to transfer electricity through its pore network [15, 54, 75, 86, 154, 169, 203, 314]. The transfer of ions or conductivity of the
concrete relies on the dissolution of ions into the pore water from hydrated cement products (such as $Na^+$, $K^+$, $Ca^{2+}$, $SO_2^-$ and $OH^-$), although Hunkelar [155] suggests that not all water in concrete will be inherently conductive. The ease with which this transfer occurs will govern the rate of corrosion [120, 121] and will depend on a number of factors, namely the interconnectivity of the pore network, moisture content and temperature. This can be broadly related to the resistive characteristics of the concrete [87, 121, 169].

The technique itself is based on the soil resistivity method [46], in which two or four electrodes are embedded into the resistive material. For reinforced concrete, a four probe electrode is utilised, which is commonly known as the Wenner probe [87, 243, 312]. A schematic of this method is shown in Figure 4.47.

![Schematic of measuring concrete resistivity using the Wenner method](image)

Figure 4.47: Schematic of measuring concrete resistivity using the Wenner method [87, 243, 312]

To measure the resistivity of the concrete, a small current is passed between the two outer electrodes (typically from an alternating current source) which invokes a current flow pattern. The resulting potential drop is measured between the two inner electrodes.
The concrete resistivity can thus be established by using Equation 4.1:

$$\rho = 2\pi a \frac{V}{I}$$  \hspace{1cm} (4.1)

where

$\rho$ = the resistivity of the electrolyte (i.e. concrete) in Ω.cm

$a$ = the spacing of the electrodes, in cm

$V$ = the potential measured between the two inner electrodes, in volts

$I$ = the current passed between the two outer electrodes, in amperes

This formula (developed by Wenner [312] in 1915) is based on a number of assumptions, namely [66, 201, 243]:

- the soil (or concrete in this case) being measured is assumed to be homogeneous and semi-infinite

- the probes are also infinitely small in comparison to the area being assessed

Practically speaking, this formula is only valid when the concrete is sufficiently large in comparison to the electrode spacing and embedded depth of the electrodes.

At this stage, it is recognised that all measurements obtained may not reflect the actual resistivity of the concrete adjacent to the steel, and thus the resistivity measured is known as an “apparent” resistivity [54, 75, 85]. As the Wenner probe is placed on the surface of the concrete, the resistivity properties being measured is that of the concrete cover. Over this depth, a resistivity profile is established, which can be influenced by a number of factors (such as moisture content, concrete properties and the homogeneity of the concrete). The probe will only measure the “average” resistivity of this profile, which may not accurately reflect the true resistivity adjacent to the steel for the purposes of corrosion risk assessment [28, 86].

Another aspect to be aware of is that the resistivity measurement only provides a guide as to the capacity of the concrete to allow corrosion, and not if corrosion has initiated or has reach its full capacity. The detection of small changes in resistivity is meaningless if
the steel is passive at the time of measurement [22, 66, 297].

However, concrete resistivity measurements can prove a useful tool in highlighting zones of “poor” quality concrete and determining the likely rate of corrosion (where corrosion has initiated). It is said to be particularly useful if conducted in conjunction with other NDT methods, in particular half-cell potential tests. However, there are many variables that can influence these results from the true resistivity, which are discussed in more detail in Section 4.5.2.

### 4.5.2 Factors Influential on Results

Although the methodology and the concept behind the resistivity test appear relatively simple, precautions must be taken prior to and during the test to ensure the accuracy of results. Millard and colleagues from the University of Liverpool have released several articles relating to factors that can introduce errors when taking concrete resistivity readings which were determined after conducting a number of experimental works and finite element analyses over the years on this topic [64, 141, 196, 198, 199, 201]. Whiting and Nagi from the Portland Cement Association [314] and the RILEM resistivity recommendation [243] also provide a good review of influencing factors. Various other reports and articles conducted between 1950’s to the present day provide additional understanding [129, 146, 155, 175, 206, 258, 267, 296]. Table 4.8 provides a summary of the main factors that may affect concrete resistivity readings.

### 4.5.3 Equipment and Methodology Adopted

Methods and equipment relevant to the current investigation are reviewed in Appendix B, which includes a review of the empirical resistivity limits adopted for the present study, in relation to the probability of corrosion and likely corrosion rates for reinforced concrete resistivity. These limits are reproduced in Table 4.9 and are based on the limits established from experimental observations made by Browne [69], Cavalier & Vassie [89] and the Concrete in the Oceans technical report #5 [276] for the Wenner method. Additional considerations have been made for the inclusion of limits established by Andrade & Alonso [23], Gonzalez et al. [138], the RILEM resistivity recommendation [243] and Rodriguez et al. [249], which have been established via the Linear Polarisation Resistance Method.
Table 4.8: Summary of factors affecting concrete resistivity readings [64, 129, 141, 146, 155, 175, 196, 198, 199, 201, 206, 243, 258, 267, 296, 314]

<table>
<thead>
<tr>
<th>Factor</th>
<th>Influence on Resistivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete properties</td>
<td>A reduction in pore size &amp; connectivity results in higher resistivity</td>
</tr>
<tr>
<td>Moisture content/</td>
<td>Complete saturation significantly reduces the resistivity;</td>
</tr>
<tr>
<td>Relative humidity</td>
<td>Very dry concrete results in very high resistivities</td>
</tr>
<tr>
<td>Temperature</td>
<td>Increase in temperature increases conductivity, lower resistivity</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>Steel decreases the measured resistivity;</td>
</tr>
<tr>
<td></td>
<td>Recommend orientating probe away from steel</td>
</tr>
<tr>
<td>Geometric Constraints</td>
<td>Depth, width &amp; boundary effects of concrete can influence resistivity significantly</td>
</tr>
<tr>
<td></td>
<td>(corrections by Millard &amp; colleagues [140, 141, 200, 201])</td>
</tr>
<tr>
<td>Chlorides</td>
<td>High concentrations of chlorides lead to a decrease in resistivity (2-3kΩ.cm less [93]</td>
</tr>
<tr>
<td></td>
<td>due to its hygroscopic nature)</td>
</tr>
<tr>
<td>Carbonation</td>
<td>Carbonated layers can lead to very high resistivity readings, where the depth of</td>
</tr>
<tr>
<td></td>
<td>carbonation will determine the margin of error (see Millard &amp; Gowers for more detail</td>
</tr>
<tr>
<td></td>
<td>[198]). Layers of varying resistivity can further compound errors</td>
</tr>
<tr>
<td>Surface contact of</td>
<td>A lack of uneven contact across the probes will lead to errors. Pre-drilling holes,</td>
</tr>
<tr>
<td>probes</td>
<td>surface pre-wetting or using detergent-soaked sponges/conductive gel on electrode tips</td>
</tr>
<tr>
<td></td>
<td>can reduce errors</td>
</tr>
<tr>
<td>Aggregate effects</td>
<td>Dense aggregates (such as basalts) may falsely increase the overall resistivity.</td>
</tr>
<tr>
<td></td>
<td>Placement of probes directly over aggregate may give erroneously high readings</td>
</tr>
</tbody>
</table>

Table 4.9: Empirical concrete resistivity limits in assessing corrosion risk [69, 89, 138]

<table>
<thead>
<tr>
<th>Resistivity (Ω.cm)</th>
<th>Likelihood of Corrosion</th>
<th>Likely Corrosion Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>ρ ≤ 5,000</td>
<td>Almost certain</td>
<td>Very high</td>
</tr>
<tr>
<td>5,000 ≥ ρ ≤ 10,000</td>
<td>Probable</td>
<td>High</td>
</tr>
<tr>
<td>10,000 ≥ ρ ≤ 20,000</td>
<td>Usually not significant</td>
<td>Low</td>
</tr>
<tr>
<td>ρ ≥ 20,000</td>
<td>Usually not significant</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

A variety of methods were used to obtain resistivity measurements using three different meters namely the Proceq4 SA RESI meter, the Tinker & Rasor SR-2 Soil resistivity meter and MEGGER® DET5/4D Earth Tester soil resistivity meter. All of these meters are commercially available. Specific methodology is discussed in Appendix B. Gross errors were found when using the SR-2 meter, therefore results from the RESI and MEGGER® have only been considered. RESI results showed a reasonable degree of correlation with those obtained with the MEGGER® using a sponge-tipped electrode, with the latter
4.5 Resistivity Survey

showing lower results and is thus more conservative. Therefore for
Section 4.5.4, MEGGER® results have predominantly been discussed for the sake of
consistency across all beams. A complete set of all results are available in Appendix C.

4.5.4 Results of Resistivity Surveys

Resistivity readings were obtained for all beams during the 2006 Winter/Spring seasons;
RESI measurements were taken during Winter and the remaining results taken during
Spring. The following discussions focus predominantly on areas where corrosion or struc-
tural distress were prominent (Bay 5 of Beam 17/4, 17/3 and 118).

On average, Beam 118 had the lowest levels of resistivity and Beam 17/4 the highest.
Trends of decreasing resistivities were observed towards the base of each beam, espe-
cially on both faces of Bay 5. Resistivities were often lower adjacent to longitudinal web
cracking and associated spalling. Areas of visible corrosion did not always comply with
the resistivities thresholds (Table 4.9), with some areas of spalling exhibiting resistivities
greater than 50kΩ.cm. The minimum recorded resistivity was 4.7kΩ.cm on the base flange
of Bay 5, Beam 118. The maximum recorded resistivity was 630kΩ.cm which was found
in a number of locations on all beams and were usually recorded using the dry drill method.

Large inconsistencies were found between results using the dry and wet drill methods.
Very high resistivities were recorded from the dry drill results which are thought to have
been affected by carbonation and resistive surface layer effects. Greater correlation ex-
isted between results obtained from the wet drill holes and the sponge-tipped electrodes,
especially for Beam 118.
4.5 Resistivity Survey

4.5.4.1 Beam 17/4

*General Observations*

As Beam 17/4 was observed to be in the best condition in comparison to the other two test beams, resistivity readings were expected to be consistently high, with lower values surrounding spalled areas on Face C of Bay 5. Figure 4.48 shows the distribution of the absolute resistivities as found using various methods. Note the large errors introduced by the SR-2 meter, with more than 95% of results being greater than 50kΩ.cm. These results were not considered further for the present study.

![Figure 4.48: Histograph of resistivity measurements obtained for Beam 17/4](image)

Resistivity values obtained using the MEGGER® meter, with all bar one value greater than 20 kΩ.cm. Over 50% of results ranged between 20 and 50 kΩ.cm; the minimum resistivity recorded was 10 kΩ.cm on Face C. Results acquired from the wet drill holes reflect similar results to the MEGGER® sponge results. Dry drill results had a tendency to be high, with over a third of results being greater than 100kΩ.cm. The latter results were considered to be affected by carbonation and resistive surface layer effects, and have been excluded from review herein. Generally, resistivities for Face C were lower than those for Face D.
4.5 Resistivity Survey

Equiresistance Plots, Thresholds and Visual Observation

Equiresistance plots were completed across the webs of both faces for Bays 1, 2, 5 and 6, using data collected from the RESI and MEGGER® meters. Full records of these results are included in Appendix C. The equiresistance plots presented are idealised and based on equal grids, which do not take into account the parabolic drape of the prestressing strands. As such, these contour plots are to be used as a guide only.

Figure 4.49 shows the MEGGER® equiresistance plot for both faces of Bay 5, which includes a photograph over which the location and value of the resistivity measured were overlayed. Referring to Face C (Figure 4.49a), the resistivities on this face were consistently greater 30kΩ.cm, indicating a low or negligible corrosion rate (Table 4.9). Higher readings were observed along the upper and lower flanges. One low resistivity value was recorded between two corroding ligatures adjacent to Diaphragm (v) (denoted by Point A), with a minimum of 10 kΩ.cm. Note the steep resistivity gradients. Note also that a similar low reading was not obtained for the adjacent corroding ligatures.

Resistivities recorded on Face D of the same bay were similar to those from Face C with the exception of the 10kΩ.cm reading (Figure 4.49b). Instances of spalling or corrosion were not observed across this face. Regarding the remaining bays tested for Beam 17/4, consistently elevated resistivities were observed, the majority greater than 50kΩ.cm. The trend of very high resistivities along the upper and base flanges was again evident, even in areas with surface laitance or poor-quality concrete (see Section 4.2).

4.5.4.2 Beam 17/3

General Observations

Resistivity results for Beam 17/3 were anticipated to be lower in zones of spalling and visible corrosion. Decreasing resistivities were observed towards the base of the beam and towards defects in Bay 5. However, the majority of the resistivity values fall between 20 and 90kΩ.cm, indicating a low or negligible corrosion risk according to Table 4.9. Figure 4.50 shows the result distribution for each method. Results from the SR-2 meter have been excluded due to error. Overall, the resistivities were lower than those measured on Beam 17/4.
Results from the MEGGER® meter show 75% of results fell between 20 and 80 kΩ.cm. Almost 15% of results fall were less than 20 kΩ.cm, indicating an increase in the likelihood of corrosion (Table 4.9). Results for Face D were widely distributed, whereas those for Face C did not exceed 70 kΩ.cm. Wet drill results show a marked reduction in resistivity.
4.5 Resistivity Survey

Figure 4.50: Histogram of resistivity measurements obtained for Beam 17/3

in comparison to other methods, with approximately 95% of results ranging between 0 and 60 kΩ.cm. Dry drill results were once again elevated as found for Beam 17/4. Resistivities for this method were generally less on Face D than Face C.

Equiresistance Plots, Thresholds and Visual Observation

Equiresistance plots were completed across the webs of both faces for Bays 4, 5 and 6, using data collected from the RESI and MEGGER meters. Results are presented for Bay 5 only; detailed results are found in Appendix C.

Figure 4.51 show the equiresistance plots for both faces of Bay 5. Lower resistivity values were measured on average. Resistivity values for Bays 4 and 6 show slightly elevated resistivities in comparison with Bay 5, but had a smaller degree of variation. Resistivities averaged approximately 65 kΩ.cm. Values towards the top of the beam ranged between 50 and 100 kΩ.cm, especially along the soffit of the top flange. Decreasing resistivities towards the base of the beam were noted on both faces. Point A on Face C and D (Figures 4.51a and 4.51b respectively) showed lower resistivities adjacent to longitudinal web cracking, correlating to a low to high likely corrosion rate. There were no visible signs of corrosion at this location and the majority of the concrete surface was hard and smooth.
(with the exception of past investigations observed on Face D). These results tended towards a “high” likely corrosion rate category (Table 4.9), although many of these results from this region ranged between 10 and 20 kΩ.cm. Note the low resistivity value adjacent

(a) Face C (kΩ.cm)

(b) Face D (kΩ.cm)

Figure 4.51: Equireresistance plot and visual overlay for Bay 5, Beam 17/3
to poorer quality concrete at Point B on Face D (Figure 4.51b), indicating a high likely corrosion rate.

An instance of rust staining was observed at Point B on Face C (Figure 4.51a). Here resistivity values averaged 20 kΩ.cm, which represents a low corrosion rate or an insignificant corrosion risk. Note also the level of spalling and corrosion initiation on a shear ligature at Point C on Face D (Figure 4.51b). Adjacent to this spall, resistivity values were remarkably high (exceeding 200kΩ.cm), indicating a negligible corrosion rate. This observation was not isolated, as several other readings showed high resistivities. Interestingly, similar resistivities were absent on Face C. It is thought perhaps that carbonation had some involvement on these readings, of which high carbonation levels were recorded in this region (see Chapter 5).

4.5.4.3 Beam 118

General Observations

Similar results were anticipated for Beam 118 as was found for Beam 17/3. However resistivities were found to be significantly lower than both Beams 17/4 and 17/3. The shift was evident from the resistivity distribution shown in Figure 4.52. The majority of results ranged between 0 and 60kΩ.cm, and approximately 25% of results for all methods were less than 20 kΩ.cm, indicating a higher likely corrosion risk. A minimum resistivity of 4.71kΩ.cm recorded along the base flange of Face C, Bay 5, which relates to a very high likely corrosion rate (Table 4.9).

All data sets followed a standard distribution, with the exception of the RESI meter results where over 10% of results lay between 90 and 100 kΩ.cm. However, almost half of the results from this data set was less than 30 kΩ.cm. Approximately 10% of all results were less than 10 kΩ.cm (a high likely corrosion rate). Similar findings were noted for the MEGGER® results. Results obtained from the wet drill holes were more dramatic, with almost 90% of results less than 30kΩ.cm and 20% less than 10kΩ.cm. Dry drill results also indicated low resistivities, although highly elevated values still accounted for 10% of the results. Face D values were noticeably less than those observed for Face C.
Figure 4.52: Histogram of resistivity measurements obtained for Beam 118

**Equiristance Plots, Thresholds and Visual Observation**

Equiristance plots were completed across the webs of both faces for Bays 4 and 5, using the RESI and MEGGER® meters. Resistivity readings were also obtained for Face C of Bays 1 and 2 using the RESI meter. Results for Bay 5 will now be discussed; full results are found in Appendix C.

Figure 4.53 shows the equiristance plots for both faces of Bay 5. This bay was noted to have the lowest resistivities on average, with only 25% of readings recorded for the other bays less than 40kΩ.cm. All bays showed a general decrease in resistivities towards the base of the beam. Results for Bay 5 showed reduced resistivities adjacent to longitudinal cracking, with resistivities closer to Diaphragm (iv) approaching 10kΩ.cm for both faces (refer to Point A on Figures 4.53a and 4.53b). This is indicative of a high to very high likely corrosion rate (Table 4.9). Lower resistivities between 5 and 17kΩ.cm were observed to follow spalling on Face D, however one isolated resistivity measurement of almost 60kΩ.cm was noted at Point B (Figure 4.53b).
Resistivities between 5 and 10 kΩ·cm were observed along the base of the beam for both faces. Note in particular on Face C the very low resistivity of 5 kΩ·cm at Point B (Figure 4.53a), indicating that corrosion was almost certain and likely to be occurring at a high rate.

(a) Face C (kΩ·cm)

(b) Face D (kΩ·cm)

Figure 4.53: Equiresistance plot and visual overlay for Bay 5, Beam 118
rate in this vicinity (Table 4.9). In comparison, a very high resistivity was recorded at Point C in Figure 4.53a, indicating a negligible corrosion rate. It is unlikely that carbonation depths at this location influenced this result. Interestingly, the concrete in these regions was in reasonable condition and very hard.

### 4.5.5 Comment on Factors Influencing Results

It has already been realised that carbonation effects may have influenced resistivity readings (see Section 4.5.4.2). In addition, it is thought that factors such as seasonal effects, steel locations and geometric constraints may have contributed to possible errors. An example is provided in Figure 4.54, which shows two sets of readings obtained using the MEGGER® meter a few days apart. The temperature and relative humidity were very similar; a 2°C temperature difference was recorded between the dates and relative humidities were almost identical. As shown by Points A and B, vast differences were recorded, with interpretations ranging from a low to a high corrosion rate for tests taken at the same location. Gowers and Millard [141] suggest that a 1°C increase in temperature will lead to a reduction of 3kΩ.cm in the resistivity. If this is the case, then differences of 6kΩ.cm should be observed between the results, however this is clearly not the case.

Difficulties were also encountered in avoiding reinforcement. The parabolic trajectory of the tendons combined with the fixity of the resistivity probe (i.e. electrode spacings were not able to be altered) and the shear ligature spacing meant that the close proximity of the electrode to the steel was unavoidable. Millard and Gowers [140, 141] estimate that errors may result in an underestimation of the true resistivity by up to two times.

Overestimation of true resistivities is perhaps more likely for the present study due to the thin web section and small flange depths, combined with the assumption that the concrete is to be semi-infinite for the Wenner principle to apply. Referring to resistivity error charts prepared by Gowers and Millard [141], it is speculated that errors due to these two boundary constraints could overestimate the true resistivity by up to two times.

Considering these effects, the experimental resistivity readings obtained must be treated with some suspicion, especially in light of the large differences observed in some places and the lack of reproducibility in others. Further discussion of resistivity results is found
4.5 Resistivity Survey

27 October 2006

31 October 2006

Figure 4.54: Example of environmental influences on resistivity measurements (Face C of Beam 17/3, kΩ.cm)

in Chapter 7 in relation to the physical condition of the steel and estimated corrosion rates.

4.5.6 Summary of Resistivity Surveys

In summary, it was found that the method for measuring concrete resistivity shows variability of results. On average, Beam 17/4 had the highest resistivities, and Beam 118 the lowest. Whilst there is a trend for lower resistivities towards the base of both Beams 17/3 and 118, there are some inconsistencies with high resistivities co-existing with those that are low. It is known that factors such as steel location, the geometric constraints of the beam and carbonation effects were likely to have induced errors in the results, however it is difficult to quantify and correct for these errors based on the current literature. In order to assess the accuracy of this technique, results from this section are discussed in relation to the condition of the steel in Chapter 7.
Chapter 5

Destructive Testing

5.1 Introduction

In some cases, non-destructive testing is considered insufficient to provide complete information on the condition of a structure. Therefore it is deemed necessary to conduct further investigations which are likely to require some form of repair after the tests are complete. For the purposes of the present investigation, such tests will be defined as “destructive” tests; these tests include load testing, chloride profiling and the observation of carbonation depths. These tests have previously been conducted on the Sorell Causeway Bridge whilst in service. However, due to limited information and changes in project requirements the three test beams have been subjected to a more intensive program of investigations involving destructive testing. Section 5.2 discusses the load performance of each beam tested to destruction; Section 5.3 presents information pertaining to chloride profiles from both cross-sectional and longitudinal concrete samples; and Section 5.4 provides an overview of detailed carbonation profiles obtained for both cores and beam cross-sections.

5.2 Load Testing

5.2.1 Introduction

The key components to ensuring the serviceability of a structure is to have an understanding of its condition and its load-carrying capacity. Conducting insitu load testing can facilitate understanding of this latter component, and this technique has become in-
creasingly popular over the last few decades [76].

The majority of insitu load tests are non-destructive. Static or dynamic loads are applied to the whole structure or span but only within the structure’s linear-elastic load range so as to avoid permanent damage. This form of testing has previously been conducted on the Sorell Causeway Bridge (discussed in Chapter 3). However, this approach did not determine the maximum load the structure would be able to withstand before collapse, and gave no insight into the serviceability of those beams affected by severe corrosion. It was this uncertainty that motivated DIER to request full-scale load testing for this project.

Whilst full-scale load testing was not feasible for the bridge whilst it was in service, it was deemed appropriate to test the individual beams to failure in the laboratory thus providing information on:

1. the ultimate load-carrying capacity of each beam
2. the compliance of the beams with respect to their designed capacity
3. whether corrosion had impaired the performance of the beams and to what degree, and
4. the relative accuracy of previous load tests and assessments conducted and recommendations provided

Based on these requirements, a load test rig was assembled in the Structures Laboratory at the University of Newcastle and Beams 17/4, 17/3 and 118 were all tested to destruction. The following sections discuss the relevance of corrosion on load-carrying capacity for reinforced and prestressed concrete, the original design requirements for the beams and loading results obtained for each beam. Particular attention has been paid to the relationship between the condition of the beams and the load capacities.

5.2.2 A Review of the Impact of Corrosion on Flexural Strength

The use of load testing for condition analysis of a bridge stems back many decades[117]. Load tests have traditionally involved proof loading to determine whether the structure would be able to sustain the standard legal loads of the day. Load tests have also been
used to assess load distributions, member stresses, whole-structure dynamic responses, defect detection, and ultimate capacity studies [216].

It is known that corrosion can have a significant impact on the flexural strength of a reinforced, prestressed concrete member [18, 76, 171, 262, 280, 293, 318]. In general, the greater the amount of corrosion suffered by the member, the greater the reduction in flexural strength. This can be attributed to the reduction of the cross-sectional area of the reinforcement, the deterioration of the bond strength at the concrete and steel interfacial zone, the reduction in reinforcement and member ductility (resulting in lower deflections), and a reduction in the effective depth of the concrete member from section loss attributable to spalling [7, 14, 81, 83, 95, 158, 179, 222, 250, 291, 305].

The factor thought to be most influential in determining flexural strength is bond strength at the concrete/steel interface [17, 53, 95, 119, 180, 305]. The contribution of localized corrosion to the reduction in flexural strength is generally considered to be relatively insignificant for normally reinforced structures, although it does significantly impair the ductility of the steel. The formation of corrosion products usually assists in the reduction of flexural strength. This can occur in two ways:

- the expansion of the corrosion products causes unsustainable tensile stresses on the concrete (leading to longitudinal cracking along the reinforcement as a result of the reduction of the constrictive nature of the concrete [14, 179], and

- the corrosion products act as a form of lubricant, reducing the adhesion and cohesion at the rebar surface thus reducing the transfer of stresses from the steel to the concrete (and vice versa) [7, 14, 18]

The relationship between the amount of corrosion products formed and the subsequent loss of steel cross-section in relation to flexural capacity remains the subject of many investigations. Several whollistic studies have been conducted on the serviceability and safety of reinforced concrete structures undergoing corrosion [191, 264, 292–294]. Other studies have made attempts to correlate the amount of corrosion with reductions in load capacity based on experimental studies. These have been summarised in Table 5.1, which show that there is a wide degree of variability amongst results. In addition, other studies
have shown that a small amount of corrosion can lead to an increase in bond strength (and subsequent increases in flexural strength) due to an increase in bar roughness and frictional characteristics [7, 18, 95, 112, 179, 262].

Table 5.1: Examples from the literature regarding the corrosion/flexural capacity relationship

<table>
<thead>
<tr>
<th>Reference</th>
<th>Amount of Corrosion</th>
<th>Effects on Load Capacity</th>
<th>Effect on Deflection</th>
<th>Effect on Bond Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Almusallam et al. [14]</td>
<td>5% overall corrosion 25% overall corrosion</td>
<td>25% reduction 60% reduction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Amleh &amp; Mirza [18]</td>
<td>4% mass loss 17.5% mass loss</td>
<td>-</td>
<td>-</td>
<td>9% reduction 92% reduction</td>
</tr>
<tr>
<td>Auyueng et al. [53]</td>
<td>2% diameter loss</td>
<td>-</td>
<td>-</td>
<td>80% reduction</td>
</tr>
<tr>
<td>Chung et al. [95]</td>
<td>≥ 2% diameter loss</td>
<td>decrease (unspecified)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>El Maaddawy et al. [112]</td>
<td>15% mass loss</td>
<td>-</td>
<td>increase in deflection less impact on deflection</td>
<td>-</td>
</tr>
<tr>
<td>Mangat &amp; Elgarf [179]</td>
<td>10% reduction in cross-section</td>
<td>25% reduction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Takagishi et al. [273]</td>
<td>10% overall corrosion</td>
<td>20% reduction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Torres-Acosta et al. [281]</td>
<td>14% reduction in radius</td>
<td>32% decrease in stiffness</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Uomoto &amp; Misra [290]</td>
<td>1-2/4% mass loss</td>
<td>17% reduction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Uomoto et al. [291]</td>
<td>Beams with corrosion</td>
<td>67-96% reduction compared to non-corroded beams</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Yoon et al. [324]</td>
<td>≥ 3% mass loss</td>
<td>decrease (unspecified)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Corrosion can also influence the mode of failure experienced by reinforced and prestressed concrete beams. Tachibana et al. [272] observed from their experimental work that corrosion damaged beams tested to destruction failed in bond shear (or slippage) in a brittle manner. Similarly, Yoon et al. [324] observed that beams subjected to increasing corrosion failed in bond shear. It should be noted, however, that the test beams in these studies were fabricated without shear reinforcement. Al-Sulaimani et al. [7], Rodriguez et al. [250] and Uomoto et al. [291] tested shear reinforced concrete beams to destruction and found that those with increasing amounts of corrosion were more likely to fail in shear rather than flexural bending. With respect to deflections, it appears throughout the literature that the extent of deflection is also governed by the amount of corrosion. El Maaddawy et al. [112] suggest that where corrosion levels are less than 15%, mid-span deflections will increase; the opposite is true for increasing levels of corrosion. Similar findings are recorded by Almusallam et al. [14], Mangat [180], and Rodriguez [250]. This is most likely due to the reduction in ductility with increasing severity of corrosion and reduction in steel cross-sectional area. Severe pitting has also been attributed to catastrophic failures of structures affected by corrosion (especially prestressed structures) where the ductility of the steel is reduced due to corrosion [60, 222, 321].

5.2.3 Ultimate Load Estimation of the Test Beams

Prior to conducting load tests, design and loading information was reviewed. This involved the determination of the material properties of the beams (such as tensile and concrete compressive strengths) and the examination of the ultimate flexural capacity of the beams in order to establish an upper load limit and for future comparisons. The following sections review this information.

5.2.3.1 Material Properties

It is important to confirm the properties of the materials used in the construction of the three test beams in order to assess whether the beams were manufactured to specifications and to estimate ultimate loads. The following sections outline the assessments of materials used in constructing the beams.
5.2 Load Testing

**Tensile Strengths**

Design drawings S-509-45 and 47 [110] specify that all post-tensioning strands are to be hard-drawn wire with an ultimate tensile strength of 110 Tons/square inch and elongation not greater than 0.1% determined at the proof stress, specified at 70% of the ultimate strength. This translates to an ultimate stress of approximately 1670 MPa.

Tensile tests were carried out on a random number of post-tensioning strands that were in good condition. The majority of these strands were retrieved from the upper tendon of Beam 17/4. One good condition and two poor condition strands from Beam 118 were also tested. The stress-strain relationships from the tensile tests are shown in Figure 5.1, and all show relatively similar profiles. On average, the ultimate load and stress achieved by the strands in good condition was 36.2 kN and 1840 MPa respectively. As seen in Figure 5.1, the majority of the strands exhibited a ductile nature, although half of the strands failed outside of the extensometer. The two severely corroded strands from Beam 118 show a drastic reduction in ductility, however the ultimate strength was not significantly impaired.

![Stress/strain relationship of prestressing strands](image)

Figure 5.1: Stress/strain relationship of prestressing strands
Concrete Compressive Strengths

Drawing 509-S-47 [110] specifies that the concrete in the precast girders is to have a 28-day strength of 5000 psi, or approximately 34 MPa, an unusually high strength for this era. Strengths in the order of 3500 psi were normally specified in the early 1950’s [71]. Whilst there is no data to record what these strengths were, it is known in the literature that concrete strengths will continue to increase as the cement continues to hydrate beyond 28 days. To determine the current day concrete strength, a number of cores retrieved from all three beams were cut into smaller sections free of reinforcement and tested for compressive strength. A summary of these results is shown in Figure 5.2, which has been adjusted for the specimen size in accordance with AS1012.9 [35]. Overall, the compressive strengths for the three beams averaged 61.1 MPa, a considerable improvement on the recommended 28-day strength. Individually, Beam 17/4 averaged 62.3 MPa, Beam 17/3 averaged 80.1 MPa and Beam 118 averaged 50.1 MPa. The maximum compressive strength of 90.6 MPa was recorded for Beam 17/3; the minimum of 29.3 MPa was recorded for Beam 118.

![Compressive Strength of Cores](image)

Figure 5.2: Compressive strength of concrete cylinders retrieved from all beams

5.2.3.2 Estimation of the Ultimate Capacity of the Test Beams

After verifying that all three test beams conformed to geometric and reinforcement requirements, original design specifications were reviewed to allow an estimation of the anticipated ultimate load of the beams [108, 110]. Unfortunately, an earlier revision of
5.2 Load Testing

the superstructure design had only been included in the project documentation, which was subsequently superseded with the acceptance of the contractor’s alternative design. In relation to live loading, Drawing 509-S-45 [110] shows that the beams have been designed to sustain a standard A.A.S.H.O load of H20-S16 or, more specifically, a Foden Semi-trailer with an H-10 payload. A later internal report conducted by DIER [110] confirms the design loading as being “H20-S16-44-P60”. This live-load configuration is based on the 1948 AASHO Standard specification for Highway Bridges [1].

In 1994, Infratech [157] was commissioned to conduct a load test of spans 1 and 17 (Chapter 2 reviews this information). Incorporated into this report was a rigorous structural assessment conducted by the consulting engineering company, Sinclair Knight Merz (SKM). This assessment estimated that the ultimate flexural capacity of the beams was 435 kNm (which is factored and assumes an average concrete compressive strength of 50 MPa). Removing the safety factor which is assumed to be 0.8 (refer to Table 2.3 from AS3600 [36]), the ultimate capacity becomes 544 kNm. This capacity allows for 30% losses due to prestress, creep and shrinkage (estimated by SKM). These calculations have been adopted as a basis for estimating the ultimate load of the beams for the current project, and it has been assumed that all test beams shall achieve this loading when in good condition. All beam capacities determined herein are compared to this estimate.

Shear failures were not considered to be the critical failure by Infratech [157] (Chapter 3), thus all test beams were predicted to fail in flexure. However due to the location of the longitudinal web cracks in Bay 5 for Beams 17/3 and 118, a shear failure was also considered. Estimations made by SKM show that the factored ultimate shear capacity of a central girder adjacent to Diaphragm (iv) is likely to be 136kN. This correlates to an unfactored capacity of 194kN if a safety factor of 0.7 is assumed (Table 2.3 from AS3600 [36]).

Additional calculations were carried out in order to determine an upper load limit for the load test itself, which included the material properties determined earlier and excluded losses and the effect from tendon profiles. A maximum flexural capacity of 675 kNm was determined, translating to an anticipated maximum applied point load of 157 kN for a third-point load test (in accordance with ASTM C78-02 [42]). More detailed calculations are found in Appendix C.
5.2 Load Testing

5.2.4 Equipment and Methodology Adopted

An overview of the load test rig and details of the methodology used are given in Appendix B. The standard ASTM C87-02 [42] was adopted for third-point load testing of the beams, with additional guidance from Standards Australia publications AS5100.2 [37] and AS5100.7 [38]. The beams were incrementally loaded to the point of failure. During loading, load, deflection and strain characteristics about Diaphragm (iii) (located at midspan) were measured.

5.2.5 Load Test Results

5.2.5.1 General Overview

The beam in the best condition, Beam 17/4, was tested first to determine the load response and failure patterns of a beam assumed to be relatively unaffected by corrosion. Beams 17/3 and 118 were then tested in that order. During the load tests, a series of “popping” noises were heard from within the beams. The sounds commenced once loading reached approximately 50 kN, and would often continue when loading was paused, especially in the cases of Beams 17/3 and 118. Small reductions in load were registered during these events, which can be observed on the load/deflection plots (see Figure 5.3). This phenomenon was later attributed to the tension failure of the post-tensioning strands. It was also found that as loading approached its maximum, it became increasingly difficult to sustain a constant load level due to the progressive failure of the strands.

![Load-Deflection Diagrams (Average)](image)

Figure 5.3: Load-deflection plots for all three beams
Table 5.2 summarises the load test results for each beam, including the maximum load and deflection recorded at failure. As can be seen, all three beams fall short of the SKM estimate of 544 kNm, with the best condition beam (Beam 17/4) achieving 90% of the target value. The beam in the worst condition, Beam 17/3, fared the worst, achieving less than 50% of the SKM target. Figure 5.3 shows a comparative load-deflection diagram for the three beams. From examination of Figure 5.3, it is evident that Beam 17/4 exhibited far superior ductility; in contrast Beam 17/3 was extremely brittle. The maximum average loads achieved by Beams 17/4, 17/3 and 118 were 112 kN, 57 kN, and 78 kN respectively. Deflections and strains were of a similar order to its respective visual condition. In relation to design load specifications (as discussed in the Infratech/SKM report [157]), all beams fell short of the T44 vehicle design load; and whilst the beams can sustain the loads for the remaining load case scenarios identified in this report, the factor of safety is extremely low.

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>Beam 17/4</th>
<th>Beam 17/3</th>
<th>Beam 118</th>
</tr>
</thead>
<tbody>
<tr>
<td>SKM maximum moment, $M_u$ (kNm)</td>
<td>544</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SKM maximum shear at D(iv), $V_u$ (kN)</td>
<td></td>
<td>194</td>
<td></td>
</tr>
<tr>
<td>Maximum moment achieved, $M^*$ (kNm)</td>
<td>483</td>
<td>243</td>
<td>333</td>
</tr>
<tr>
<td>SKM maximum point load, $P_u$ (kN)</td>
<td></td>
<td>126</td>
<td></td>
</tr>
<tr>
<td>Maximum point load achieved, $P^*$ (kN)</td>
<td>112</td>
<td>57</td>
<td>78</td>
</tr>
<tr>
<td>% of SKM Estimate</td>
<td>89%</td>
<td>45%</td>
<td>61%</td>
</tr>
<tr>
<td>% of Beam 17/4</td>
<td>-</td>
<td>50%</td>
<td>69%</td>
</tr>
<tr>
<td>Predicted mode of failure</td>
<td>Flexure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual mode of failure</td>
<td>Flexure</td>
<td>Shear</td>
<td>Shear</td>
</tr>
<tr>
<td>Location of Failure</td>
<td>Dia (iii)</td>
<td>Dia (iv)</td>
<td>Dia (iv)</td>
</tr>
<tr>
<td>Deflection at Failure (mm)</td>
<td>433</td>
<td>102</td>
<td>271</td>
</tr>
</tbody>
</table>
5.2 Load Testing

The following sections offer further insight of the load tests for each beam, including load-deflection plots and strain progression over time. It should be noted that strain results for Beams 17/3 and 118 are not strictly accurate, as the strain gauge placement was at midspan which is not where the respective failures occurred.

5.2.5.2 Beam 17/4

The load test of Beam 17/4 took place over two days, which explains the dip in the load-deflection diagrams at a deflection of approximately 200mm shown in Figure 5.4. Some divergence was noted in the measured deflections between Faces C and D; this may have occurred due to the out-of-plane rotation experienced by the beam during loading, indicating a lack of symmetry instigated either from the loading positions or from the features of the beam itself. The eccentricities induced from this asymmetry are likely to have influenced the measured ultimate load, although the effect was likely to be minimal. Figure 5.5 shows the deflection patterns of Beam 17/4 close to its failure load, achieving a maximum deflection of 433 mm at centre span.

![Beam 17/4 Load-Deflection Diagram (P1, P2)](image)

Figure 5.4: Load-deflection plot for Beam 17/4

The maximum average load of 112 kN was reached within the first stage of load testing, which was calculated from the combination of the two load plots measured for the beam (Figure 5.4). By the end of the loading process, approximately 10 strands were estimated
5.2 Load Testing

Figure 5.5: Deflection characteristics for Beam 17/4 under maximum load

to have failed in tension, although more may have failed judging by the small peaks observed in the load-deflection diagram in Figure 5.4. The two larger peaks observed on this plot show the unloading process of the beams due to time delay in the load test, and that the loads recorded for each load cell cancel out. It is interesting to note, however that there is no recovery in the deflection of the beam recorded at this point in time, however the opposite is true at the conclusion of the test. In relation to the maximum load achieved by Beam 17/4, it appears that the SKM flexural capacity overestimates the capacity of the beams by approximately 10% (assuming that the condition of the steel in Beam 17/4 is unaffected by significant corrosion).

Strains measured on both Face C and D exhibited similar patterns, as shown in Figure 5.6. Maximum tensile strains of approximately $1.7 \times 10^{-2} \epsilon$ were recorded along the base of the beam, although low, compressive strains were recorded across the middle and base of Face C, Bay 4.
5.2 Load Testing

Figure 5.6: Strains measured at midspan for Beam 17/4

The failure mode for Beam 17/4 was in flexure and is shown in Figure 5.7. Large vertical cracks (10-30mm) formed either side of Diaphragm (iv), with the worst cracking being located on Face C. The cracks emanated away from this point at spacings approximate to the placement of the shear ligatures, with crack widths reducing with increasing distance from Diaphragm (iii). As expected, crack widths were most significant towards the base of the beam.

Figure 5.7: Beam 17/4 crack pattern development at failure
5.2.5.3 Beam 17/3

Beam 17/3 failed catastrophically in shear which stemmed from Diaphragm (iv) and the longitudinal cracking observed in Bay 5, as shown in Figure 5.8. Diagonal cracks formed on both faces of Bay 5 adjacent to Diaphragm (iv) just prior to failure. Failure occurred when the majority of prestressing strands from the lower tendon had failed and the base flange close to the centre of the bay split vertically, causing the section to drop suddenly. It was estimated that seventeen strands failed during the test, some of which occurred whilst the load was stationary. On both Faces C and D, the longitudinal web crack widths were greater than 50mm at the failure point, with the appearance that the lower flange was about to detach from the rest of the beam (Figure 5.8).

![Figure 5.8](image)

Figure 5.8: Overview of Beam 17/3 failure

Figure 5.9 shows a close-up of strands from the lower tendon within the failure zone. The condition of the strands was poor, with pitting evident. Some strands were found to have failed in a ductile nature (denoted by Point A) whilst others failed by brittle fracture creating jagged edges (Point B). The physical condition of the steel is discussed further in Chapter 6.

Load-deflection and strain plots are shown in Figures 5.10 and 5.11 respectively and shows a significant reduction in strength and ductility in comparison to Beam 17/4. The overall maximum load achieved was just 57 kN, which is less than half of the expected load from SKM. Maximum tensile strains of between 3.0 and $4.0 \times 10^{-4}$ were recorded along the base flange; one compressive strain of similar magnitude and location was also noted on
5.2 Load Testing

Figure 5.9: Close-up of failed strands from Bay 5, Beam 17/3

Face D. The maximum deflection recorded at midspan was 73mm on average (deflections measured for Face C and D were in general agreement); using geometry, the projected deflection at the failure point, Diaphragm (iv), is 103mm. It should be noted, however, that at the point of failure the beam was prevented from fully collapsing by the installation of supports under the failure point, therefore the maximum deflection recorded relates to the termination of the load test.

Figure 5.10: Load-deflection plot for Beam 17/3
5.2 Load Testing

5.2.5.4 Beam 118

Beam 118 also failed in shear through the Diaphragm (iv)/Bay 5 interface, although failure was prolonged rather than the sudden collapse observed for Beam 17/3. The overall failure mechanism is shown in Figure 5.12a, and failure patterns induced in the web of Bay 5 are shown in Figure 5.12b. Consistent with Beam 17/3, significant cracking and steel section losses were noted at the failure site. Crack widths were not as pronounced as those observed for Beam 17/3, with diagonal cracks stemming from Diaphragm (iv) exhibiting widths of approximately 30mm; longitudinal cracking opened up to widths of no more than 15mm. Approximately 33 strands failed during the load test, representing almost all strands from both tendons. Some failures occurred simultaneously, with up to 3 or 4 strands failing at one time. Figure 5.13 shows a close-up of the strand group from the upper tendon, showing both ductile and brittle fractures similar to that observed for Beam 17/3.

The drawn out nature of the test and strand failures are evident in the load-deflection plot (Figure 5.14). Note the slightly elevated loads recorded for P2, which was adjacent to the failure location. The peak average load recorded for Beam 118 was 78 kN, with a midspan deflection of 193mm recorded. The corresponding deflection at Diaphragm (iv) is estimated to be approximately 270mm, although the load test was terminated prior to

Figure 5.11: Strains measured at midspan for Beam 17/3
5.2 Load Testing

The beam collapsing entirely. The highest recorded tensile strain was on Face C along the base flange of Bay 3, which recorded a maximum of approximately $3 \times 10^{-3} \epsilon$ (Figure 5.15). The maximum compressive strain was also on Bay 3, registered approximately $4 \times 10^{-3} \epsilon$ across the top of the web on Face C. Interestingly, tensile strains were also measured in similar locations on Bay 4. A jump in all strain readings was recorded approximately half-way through the test; this coincides with a pause in the test.
5.2 Load Testing

5.2.5.5 Other Observations

After conducting the load tests, the beam sections were once again visually inspected for reinforcement placement compliance and corrosion effects. An important discovery across all beams was the poor splicing detail noted for the base longitudinal reinforcement over the diaphragms. As noted in drawing number 509 S-48 [110] (reproduced in...
part in Figure 5.16a), splice bars of approximately 350mm length were specified to span across the length of the diaphragms to connect prefabricated reinforcement cages from each bay. An example of poor splice detailing is given in Figure 5.16b. Reviewing the current standard AS3600 [36] and a state of the art review of bond stresses conducted in 1966 [5], it appears that the observed splice detail may have been of insufficient length to adequately transfer stresses between the steel and the concrete. This may have had implications on the overall flexural strengths achieved by the beams.

(a) Splice specifications from Drawing 509 S-48

(b) Actual splice length over Diaphragm (iv), Beam 17/3

Figure 5.16: An example of poor splice detail over diaphragms
5.2 Load Testing

5.2.6 Summary

Load test results showed that, in its simply-supported state, all three beams did not achieve the flexural capacity estimated from SKM’s structural assessment [157], with Beam 17/4 (in the best condition) falling 10% short of this target. These beams did not comply with the T44 load case, with little reserve of capacity for other load cases outlined in the Infratech report [157]. It was also apparent that the bridge was heavily reliant on load transfer through the diaphragms, which was compromised by poor splicing detail.

Corrosion had significantly impaired the capacity of Beams 17/3 and 118, with Beam 17/3 only achieving 50% of the load of Beam 17/4. It also affected the manner in which the beams failed. Beam 17/3 failed without warning in a brittle manner in comparison to the prolonged and ductile failure of Beam 17/4. Cracking along Beam 17/3 opened up to widths of over 50mm at failure, exposing the prestressing strands whilst cracking in Beam 118 was greatest along newly-initiated diagonal cracks stemming from Diaphragm (iv). Strands at the failure point of Beam 17/4 were in good condition; the opposite was true for Beams 17/3 and 118 with significant corrosion and brittle steel failures observed. The observations made in this chapter are combined with those made in relation to steel condition in Chapter 6, with implications discussed in Chapter 7.
5.3 Chloride Profiles

5.3.1 Introduction

Many studies, dating from the 1950’s, argue that chloride contamination increases the risk of corrosion of steel embedded in concrete [21, 57, 67, 170, 230, 242]. Chloride influenced corrosion is considered to be the most insidious form of corrosion due to its association with aggressive pitting and high corrosion rates of steel reinforcement which is often not identifiable from the concrete surface. Structures typically at risk are those located in marine environments, those subjected to de-icing salts, or those with chloride admixtures incorporated into the concrete at the time of casting.

It is thought that at certain levels of chloride contamination in concrete, corrosion can initiate and continue at a significant rate. Therefore, the determination of chloride concentrations in concrete, particularly adjacent to reinforcement, during field investigations is considered crucial in assessing the condition of a structure and its remaining serviceable life. Given importance placed on chloride levels within reinforced concrete and the likely chloride contamination of the Sorell Causeway Bridge beams due to the addition of calcium chloride and the bridge’s location within an aggressive marine environment, a detailed chloride investigation was conducted on all test beams.

The following sections provide the current understanding of chloride involvement in the corrosion process, the chloride threshold at which corrosion will theoretically initiate, and factors affecting contamination. An overview of the results found for each beam for both cross-sectional and longitudinal chloride levels is also given.

5.3.2 Chloride Transportation to Embedded Reinforcement

In order for chloride-induced corrosion to take place, chloride ions must be present at the depth of the reinforcement. This requires the transportation of chloride ions from a chloride-rich source. For the Sorell Causeway bridge, representative chloride sources include the proximity to a marine environment and the presence of the concrete setting accelerant calcium chloride (see Chapter 2). In the case of soluble chlorides, chloride salts must ingress through the concrete from the surface to the embedded steel.
5.3 Chloride Profiles

This transportation is governed by several mechanisms [131]:

- **Diffusion:** the chlorides will move through the concrete pore network due to a concentration gradient (i.e. chlorides in high concentrations will move towards areas of low concentration). This can only take place in pores that are totally or partially water filled [242]

- **Migration:** chloride ions will move through due to the formation of an electric field (i.e. will be drawn to areas of positive charge due to the negative charge of the chloride ions)

- **Water Flow:** chlorides contained in water will be forced through the pores of the concrete due to the movement of water into the concrete (i.e. a coastal structure subjected to tidal movement or splash zones)

Most published studies have concentrated on the transportation of chlorides through diffusion. This is because most chlorides will ingress from external sources and the addition of chloride additives are now banned or restricted [30, 57, 67, 209, 230, 303]. However, due to the age of the Sorell Causeway Bridge, both the effects of environmental chlorides and calcium chloride need to be considered in this present study.

The movement of chloride ions in a structure located in a marine environment or subjected to de-icing salts will generally subscribe to Fick’s second law of diffusion [63, 131]. Chloride diffusion is a complex process which involves variable chloride concentrations and there is gradual increase in concentration of chlorides in the concrete rather than an advancing chloride front.

Fick’s second law of diffusion in relation to chloride ingress is stated in equation 5.1:

\[
\frac{d[Cl^-]}{dt} = D_c \left( \frac{d^2[Cl^-]}{dt^2} \right)
\]

Equation 5.1 can be integrated to provide the chloride concentration with depth into the concrete, as shown by equation 5.2:

\[
C_s = C_c \left( \frac{x}{2\sqrt{D_c t}} \right)
\]
5.3 Chloride Profiles

The concentration of chloride ions that has diffused from the concrete surface with increasing depth has been simplified in the literature (taking into account Fick’s second law of diffusion) and is shown in Figure 5.17. The plot in red represents chloride profiles that may be anticipated if chloride ions have been deposited on the surface of the concrete and have diffused inwards towards the reinforcement [21, 221]. A peak chloride concentration is likely to be observed away from the concrete surface, as environmental actions such as tidal movements and precipitation may result in the reduction of chloride levels on the surface. From this point, the plot is defined by decreasing chloride concentrations with increasing depth. This plot assumes that there are no chloride additives present in the concrete. Where additives are present, the profile is anticipated to be substantially different, with irregular and consistently elevated chloride levels with increasing depth [230] as depicted by the blue plot line in Figure 5.17.

Figure 5.17: Simplified graph showing the variation of chloride concentration with increasing concrete depth due to surface deposition (red) and the presence of a chloride admixture (blue)
5.3 Chloride Profiles

Whilst Equation 5.2 and the red plot line in Figure 5.17 represents the ideal diffusion of chloride ions through the concrete, there are many variables within the concrete itself that can alter the rate of diffusion. This includes the presence of cracks, changes in the concrete properties (such as the pore network), and moisture content [67].

5.3.3 The Chloride Threshold & Corrosion Initiation

As previously identified in Chapter 3, the exact mechanism regarding the involvement of chloride ions in the destabilisation of the passive oxide film of the steel (or depassivation) remains unknown [86, 131, 182]. Current-day thinking hypothesises that chloride ions will accumulate at the steel/concrete interface, and when a certain concentration of chlorides has been reached, the passive oxide film becomes unstable, will break down locally and corrosion can thus initiate [2, 21, 63, 148, 275, 303].

There still exists some confusion as to the exact definition of the chloride threshold. Glass and Buenfeld [132] define the chloride threshold as being “the quantity of chloride at the steel that is necessary to sustain local passive film breakdown and hence initiate the corrosion process”, however this begs the question of what that level is. Some researchers argue that at point of the film breakdown corrosion or rust spotting is visible on the steel surface [139, 148]. Alternatively, corrosion initiation can be represented by a shift in the corrosion potential or by reaching a certain level in the corrosion current [16, 134]. For the purposes of this research, a visual representation was adopted.

Many laboratory and field studies have been carried out to establish a value for the chloride threshold. Recent reviews of such data has been examined by Glass and Buenfeld [134], Alonso et al. [16], and most recently by Ann and Song [29]. These studies show that the threshold for corrosion initiation can vary between 0.17 to 2.5% Chloride by weight of cement (% Cl\(^{-}\) (cement)) for concrete structures (both field and laboratory results). This takes into account the total chloride concentration, incorporating both free and bound chlorides, however it is commonly thought that only free chlorides participate directly in the corrosion process [31, 153, 209, 285]. Alternatively, the threshold may be represented by a chloride/hydroxyl ion ratio, of which it is stated that corrosion may initiate when this ratio is greater than 0.6 [21, 139, 148, 164]. This method is thought to be inappropriate for reinforced concrete and difficult to measure [16, 156, 279]; it also
fails to take into account the effect from bound chlorides (see Section 5.3.4).

Several authors discourage the adoption of universal chloride threshold limits [74, 205, 279]. However, throughout the literature and standard documentation, chloride limits are being continuously specified in order to determine the corrosion risk posed by chlorides to reinforced concrete structures [2, 50, 66, 150, 298]. Whiting and colleagues have published several reviews in relation to chloride limits in concrete [313, 315]. In summary, it is recognised that structures with a chloride concentration of greater than 0.4% $Cl^-$(cement) is at risk of corrosion, however for prestressed concrete a level of 0.2% $Cl^-$(cement) is recommended. Table 5.3 provides an overview of the chloride limits generally accepted across the literature. These limits have been adopted when interpreting experimental chloride profiles obtained.

Table 5.3: Risk of corrosion due to chloride contamination [2, 50, 66, 150, 170, 230, 298, 313, 315]

<table>
<thead>
<tr>
<th>Chloride Content (by weight of cement)</th>
<th>Chloride Content (by weight of concrete)</th>
<th>Corrosion Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 0.2%</td>
<td>≤ 0.03%</td>
<td>negligible</td>
</tr>
<tr>
<td>0.2 - 0.4 %</td>
<td>0.03 - 0.06%</td>
<td>low</td>
</tr>
<tr>
<td>0.4 - 1.0 %</td>
<td>0.06 - 0.14%</td>
<td>moderate</td>
</tr>
<tr>
<td>≥ 1.0%</td>
<td>≥ 0.14%</td>
<td>high</td>
</tr>
</tbody>
</table>

5.3.4 The Phenomenon of Chloride Binding

Chloride binding has been of increasing interest since the corrosion-related effects of salt additives were first investigated around the 1960’s and 70’s [62, 63, 230, 315]. Chloride binding occurs when free chloride ions interact with the constituents of the cement paste either by a chemical reaction or through physical means such as adsorption, however the exact mechanisms are still unknown. The chlorides are subsequently removed from the concrete matrix [29, 109, 131, 211, 325]. Binding of chlorides is normally associated with chloride-based additives, such as calcium chloride, which appears to be bound at a significantly higher rate than ingressed chlorides [242, 325].
The predominant chloride binding reaction noted in the literature is the reaction of chloride ions with the tricalcium aluminate hydrates (C3A) in the cement paste to form calcium chloroaluminates (or alternatively known as Friedel’s salt) [57, 205, 209, 242]. Other constituents can form during this process, however the most predominant is Friedel’s salt [31, 68, 271].

General consensus is that only free chlorides pose a threat to the corrosion of steel reinforcement (as stated previously). Therefore, it is prudent to review the literature regarding the amount of chlorides that are bound in the concrete. Typically, chloride-binding theory states that the majority of binding occurs during cement hydration; however there are alternating views in the literature regarding how much is physically bound. Experimental results from Ramachandran [233] indicate that the majority of chlorides are bound at the completion of hydration. Cook and McCoy [99] cite experiments by Monfore and Verbeck which have similar findings with 70-90% of chlorides being bound, although they did find that a small amount of calcium chloride does remain in the mix after 28 days. However, Diamond and Lopez [109] provide experimental results showing that the proportion of chlorides are not bound as rapidly or completely in the concrete as first thought by Ramachandran.

Recent research has shown that bound chlorides may indeed be influential in the corrosion process. Research by Glass and colleagues, has shown that Friedel’s salt is pH dependent [133, 135, 136, 235]; when the pH drops below 12, this compound becomes unstable and will rapidly release the bound chlorides which dissolve back into the concrete pores as free chlorides. The drop in pH is often attributable to the progression of the carbonation front (see Section 5.4). As a result, a jump in chloride concentration may be recorded at the forefront of this pH change, and as such significantly increases the risk of corrosion. As such, the consideration of the total chloride content of the concrete is deemed to be of greater value for the purposes of the current work.

5.3.5 Factors Affecting Chloride Induced Corrosion

Whilst reinforcement corrosion is considered highly likely in a chloride-contaminated concrete environment, the time to initiation and rate of corrosion will depend on a number of factors. These factors include [57, 205, 242]:
5.3 Chloride Profiles

- **Concrete solution and pH:** the higher the pH, the greater the ratio of hydroxyl ions to chloride ions, the greater resistance to corrosion initiation

- **Diffusion characteristics of the concrete:** such as water/cement ratio, aggregate properties, interconnectivity of the pore network, curing regime

- **Concrete composition:** the greater the proportion of finer materials, the lower the risk of corrosion (e.g. adding fly ash is seen as beneficial)

- **Sorptivity of the concrete:** the ability of the concrete surface to adsorb chlorides

- **Exposure to chloride environment:** the orientation and geometry of the concrete, the frequency of application, wetting and drying periods, and the chloride concentration of the source will all influence the amount of chlorides available for diffusion

- **Binding capacity of the concrete:** the greater the binding capacity of the concrete, the smaller the percentage of free chlorides theoretically able to participate in the corrosion process

- **Carbonation:** as already discussed, chlorides can be liberated with the advancement of the carbonation front; alternatively, carbonation reduces the connectivity of the pore network, thus reducing the transportation capability of the concrete

- **Temperature:** the higher the temperature, the greater percentage of chlorides going into solution thus able to participate in the corrosion process

### 5.3.6 Equipment and Methodology Adopted

Information pertaining to equipment and methodology used for experimental work is documented in Appendix B. In summary, both core and dust samples were retrieved from pre-defined locations along Bay 5 of each beam in accordance with AS1012.14 [32] and HB-84 [150]. The location of the samples taken from Beam 17/4 vary from Beams 17/3 and 118 due to issues of access and the condition of the beams. Similar reasoning lies behind the larger number of samples taken from Beam 17/4 in comparison to the other two test beams.
5.3 Chloride Profiles

Samples were prepared using the acid-digestion method outlined in AS1012.20 [33], specifying the chloride contents with respect to concrete and cement content (cement contents were calculated based on experimental results found by DIER [110]). Cross-sectional and longitudinal profiles were determined adjacent to prestressing strands, with results presented in relation to the chloride limits shown in Table 5.3. The division between free and bound chlorides was not determined for the current study as the total chloride content was considered to be of greater value and also to allow for a direct comparison of results against previous DIER chloride investigations. Representative grout samples were also analysed for both chloride and sulphate contents.

5.3.7 Results of Chloride Profiles

Overall, results were found to be variable with no apparent trends. It was found that the chloride levels for all three beams averaged 0.7% $\text{Cl}^-$ (cement), which exceeds the threshold limit for low corrosion risk (Table 5.3). In fact, almost 75% of individual readings (cross-section and longitudinal profiles) were greater than 0.4% $\text{Cl}^-$ (cement). Chloride concentrations at reinforcement and prestress level show levels greater than the recommended limit for both prestress and reinforced concrete (that is, 0.2% and 0.4% $\text{Cl}^-$ (cement) respectively). The elevated chloride levels are likely to be attributable to the addition of calcium chloride. It was assumed that Beam 17/3 would exhibit the highest levels of chloride contamination and Beam 17/4 the lowest, however the opposite was found to be true, with the maximum chloride levels for both surface and internal profiles found in Beam 17/4. Profiles were irregular and not consistent with conventional chloride profiles detailed in the literature [21, 57, 67, 221, 230]. A background chloride level was unable to be deciphered in relation to the additive calcium chloride.

The following sections provide more detailed information pertaining to chloride levels obtained from cross-sectional and longitudinal samples for each beam. Only specific cases are highlighted in this chapter, however full results can be found in Appendix C.
5.3 Chloride Profiles

5.3.7.1 Cross-Sectional Chloride Profiles

Table 5.4 provides statistical information regarding the chloride levels obtained for all beams in relation to cross-sectional profiles, and Table 5.5 shows the division of the results for each beam in relation to the corrosion risk limits established in the literature and shown previously in Table 5.3. For the majority of profiles, chloride levels did not reduce with depth; on the contrary, elevated chloride levels were found at all depths and at times increased with depth. No clear trends were observed in relation to preferential chloride deposition between Face C or Face D, however Beam 118 does show slightly elevated levels across Face D. Similarly, geometric and orientation effects appear not to be influential, with chloride levels at various locations showing little correlation. The following sections will now make more detailed observations of cross-sectional profiles across each beam.

Table 5.4: Summary of chloride concentrations for cross-sectional profiles

<table>
<thead>
<tr>
<th>Total % Cl⁻ (cement)</th>
<th>Beam 17/4</th>
<th>Beam 17/3</th>
<th>Beam 118</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Samples</td>
<td>106</td>
<td>48</td>
<td>40</td>
</tr>
<tr>
<td>Average</td>
<td>0.8105</td>
<td>0.6159</td>
<td>0.6104</td>
</tr>
<tr>
<td>Maximum</td>
<td>4.728</td>
<td>1.5884</td>
<td>1.189</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.0935</td>
<td>0.00</td>
<td>0.056</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.601</td>
<td>0.328</td>
<td>0.348</td>
</tr>
<tr>
<td>Variation</td>
<td>0.361</td>
<td>0.111</td>
<td>0.121</td>
</tr>
</tbody>
</table>

Table 5.5: Distribution of chloride concentrations in relation to recommended corrosion risk limits stated in Table 5.3 (Cross-sectional profiles)

<table>
<thead>
<tr>
<th>Total chloride content (by weight of cement)</th>
<th>Likely Corrosion Risk</th>
<th>Distribution of Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 0.2%</td>
<td>Negligible</td>
<td>Beam 17/4</td>
</tr>
<tr>
<td>0.2 - 0.4%</td>
<td>Low</td>
<td>Beam 17/3</td>
</tr>
<tr>
<td>0.4 - 1.0%</td>
<td>Moderate</td>
<td>Beam 118</td>
</tr>
<tr>
<td>≥ 1.0%</td>
<td>High</td>
<td></td>
</tr>
</tbody>
</table>
Beam 17/4

Chloride levels recorded across the cross-section of Beam 17/4 were amongst the highest levels recorded for the three beams. Almost 80% of readings were greater than 0.4% Cl\(^-\) (cement), with a third of readings greater than 1.0% Cl\(^-\) (cement). Chloride concentrations at reinforcement level exceeded the recommended limit for prestressed concrete. As previously stated, chloride levels did not decrease with increasing depth, and chloride concentrations averaged 1.2% Cl\(^-\) (cement).

Figure 5.18 provides an overview of the chloride concentrations determined through the full width of the web adjacent to the lower and upper tendons. Chloride sample locations are shown in more detail in Appendix B. The maximum chloride level was observed on the surface of Face D at Section C, at an extremely high value of 4.7% Cl\(^-\) (cement). Remaining chloride levels remained elevated and above 1.0% Cl\(^-\) (cement) with increasing concrete depth towards the lower prestressing tendon. The lowest chloride levels were also recorded on Face D adjacent to the upper tendon close to Diaphragm (iv), however values were still in the order of 0.75% (cement). These results may have been influenced by geometric location (i.e. readings were taken adjacent to the upper prestressing tendon), however this appears inconsistent with the levels recorded for the soffit of the beam.

Two soffit locations were sampled for chloride levels, with results shown in Figure 5.19. Chloride concentrations were found to be high, consistently averaging 1.0% Cl\(^-\) (cement) consistently with depth through to reinforcement level. The data in Table 5.3 suggests that the reinforcement is at high risk of significant corrosion; however the steel at these sample locations is noted to be in good condition (see observations made in Chapter 6 and further discussions correlating steel condition with chloride levels in Chapter 7).
5.3 Chloride Profiles

Figure 5.18: Chloride profiles through the web at various locations along Bay 5, Beam 17/4 (Total chloride content)
5.3 Chloride Profiles

Although this beam was visually in the worst condition, Beam 17/3 displayed the lowest chloride levels of all beams tested. These profiles were taken adjacent to the upper tendon and therefore chloride levels may have been higher towards the base of the beam, however the majority of results were above the recommended limit of 0.2% (cement) at reinforcement level. Just over 20% of tests were greater than 1.0% $Cl^{-}$ (cement), with the majority of results lying between 0.4 and 1.0% $Cl^{-}$ (cement). The maximum concentration recorded was 1.6% $Cl^{-}$ (cement).

Figure 5.20 shows the distribution of chlorides through the web at various locations in Bay 5 adjacent to the upper prestressing tendon. Chloride levels obtained for sample #D-(b) (adjacent to Diaphragm (iv)) exhibited consistently high concentrations close to the 1.0% mark and which increased with depth.
5.3 Chloride Profiles

Figure 5.20: Chloride profiles through the web at various locations along Bay 5, Beam 17/3 (Total chloride content)
5.3 Chloride Profiles

With the exception of this sample and #A-C, chloride profiles hit a peak value and then start to subside towards the centre of the web (as per the classical chloride diffusion model for concrete). However these peak values were close to the tendon level and exceeded 0.4% $Cl^-$ (cement).

The minimum chloride concentration was recorded for sample #D-A taken from Face D, having a value of 0% $Cl^-$ (cement) at a depth between 10 and 20mm (Figure 5.21). A similar low reading was recorded for the surface of this sample. However, elevated chloride levels were recorded for the opposite face for Sample #D-B, with a maximum concentration of approximately 1% (cement) recorded at depths between 20 and 30mm. A pronounced profile is observed across this sample. It should be noted that sample #D-A was retrieved through a crack and as such only a small sample was retrieved (5g). It possible that this may have lead to the low chloride concentrations measured and perhaps should be considered with suspicion. However, it is thought that at least some percentage of chlorides would be observed (especially considering the elevated levels elsewhere), and thus this sample was not automatically discounted.

Figure 5.21: Low chloride concentrations adjacent to prestressing from Beam 17/3 (Total chloride content)
5.3 Chloride Profiles

**Beam 118**

On average, chloride levels obtained for cross-sectional profiles for Beam 118 were found to lie in between those found for Beams 17/4 and 118. As for Beam 17/3, the majority of chloride levels were between 0.4 and 1% \( Cl^- \) (cement), with only 15% of results greater than 1% (cement). This beam had the highest percentage of chloride concentrations less than 0.2% \( Cl^- \) (cement), and also showed a greater trend for higher chloride levels across Face D than Face C. The maximum recorded concentration was 1.2% \( Cl^- \) (cement) measured at reinforcement and strand level (Sample #C-3). Figure 5.22 shows the various chloride profiles determined along Bay 5. More definitive peaks were observed across all samples across Face D; increasing chloride levels were observed across Face C. All chloride levels were greater than the recommended limit at reinforcement and prestress level.

5.3.8 Longitudinal Chloride Profiles

Consecutive chloride level measurements were made adjacent to the prestressing tendon. Measurements on Beam 17/4 were taken at the lower tendon, whilst measurements on Beams 17/3 and 118 were taken at the upper tendon due to issues of access and the condition of the latter two beams. Results have been summarised statistically in Table 5.6 and result distributions in relation to corrosion risk are shown in Table 5.7. Results were relatively consistent, with the majority of levels obtained ranging between 0.4 to 1% \( Cl^- \) (cement) for all three beams. Note that Beams 17/3 and 118 have almost identical standard deviations and variations, which is lower than that for Beam 17/4. Beam 118 has a greater proportion of results greater than 1.0% \( Cl^- \) (cement).

<table>
<thead>
<tr>
<th>% ( Cl^- ) (cement)</th>
<th>Beam 17/4</th>
<th>Beam 17/3</th>
<th>Beam 118</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of Samples</td>
<td>58</td>
<td>11</td>
<td>8</td>
</tr>
<tr>
<td>Average</td>
<td>0.490</td>
<td>0.283</td>
<td>0.872</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.241</td>
<td>0.836</td>
<td>1.083</td>
</tr>
<tr>
<td>Minimum</td>
<td>0.094</td>
<td>0.336</td>
<td>0.686</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>0.280</td>
<td>0.161</td>
<td>0.162</td>
</tr>
<tr>
<td>Variation</td>
<td>0.078</td>
<td>0.026</td>
<td>0.026</td>
</tr>
</tbody>
</table>
Figure 5.22: Chloride profiles through the web at various locations along Bay 5, Beam 118 (Total chloride content)
Table 5.7: Distribution of chloride concentrations in relation to recommended corrosion risk limits stated in Table 5.3 (Longitudinal profiles)

<table>
<thead>
<tr>
<th>Total chloride content (by weight of cement)</th>
<th>Likely Corrosion Risk</th>
<th>Distribution of Results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Beam 17/4</td>
</tr>
<tr>
<td>≤ 0.2%</td>
<td>Negligible</td>
<td>21%</td>
</tr>
<tr>
<td>0.2 - 0.4%</td>
<td>Low</td>
<td>25%</td>
</tr>
<tr>
<td>0.4 - 1.0%</td>
<td>Moderate</td>
<td>52%</td>
</tr>
<tr>
<td>≥ 1.0%</td>
<td>High</td>
<td>2%</td>
</tr>
</tbody>
</table>

Figures 5.23, 5.24 and 5.25 show the longitudinal chloride profiles obtained for each beam. On average, Beam 118 registered the highest chloride levels and Beam 17/4 the lowest, however the maximum level was obtained from Beam 17/4. Results for Beam 118 were consistent, ranging between 0.8 and 1.1% $Cl^-$ (cement)(Figure 5.24); similar observations were made for Beam 17/3, however results range between 0.4 and 0.8% $Cl^-$ (cement)(Figure 5.23). Beam 17/3 showed a slight rise in levels approximately half-way along the core. Slightly higher chloride levels were measured in the sample from Beam 118 in relation to Beam 17/3, which is closer to Diaphragm (iv).

For Beam 17/4, the majority of measurements were taken along the underside of the lower tendon, with only sample #A-1 above the tendon (Figure 5.25). Elevated chloride levels were observed across samples #A-1 and #B-1 (towards Diaphragm (v)), with results ranging between 0.4 and 1% $Cl^-$ (cement). However, a drop in chloride levels was observed across samples #C-1 and #D-1 (towards Diaphragm (iv)), with most levels below 0.4% $Cl^-$ (cement). The location of these samples in relation to the geometry of the beam; the former two samples were located fully or partially in the web, whilst the latter samples were more deeply located within the base flange, with sample #D-1 showing the lowest levels (less than 0.2%).

In summary, it appears that the longitudinal profiles confirm that elevated chlorides exist at the prestressing level and that levels are relatively consistent along the length of the tendon (with small variations noted). The thickness of the base flange (in comparison to the web) appears to have been responsible for a drop in levels towards Diaphragm (iv) for Beam 17/4.
5.3 Chloride Profiles

Figure 5.23: Longitudinal chloride profiles adjacent to tendons in Bay 5, Beam 17/3
(Total chloride content)

Figure 5.24: Longitudinal chloride profiles adjacent to tendons in Bay 5, Beam 118
(Total chloride content)
5.3 Chloride Profiles

A select number of grout samples were analysed for chloride content in order to establish whether calcium chloride had been added to the grout mixture. Table 5.8 shows a summary of the average number of ionic species found in the grout. High levels of Nitrate were observed due to the acid preparation of the samples. Chloride levels were very low, equating to only 0.03% CI⁻ (by weight of the grout). From this result and the observed concentrations across the beams, it can be concluded that calcium chloride was not added to the grout. It should be noted that equivalent levels of sodium were also found in the grout, which may indicate the progression of NaCl to the level of the tendons.

Figure 5.25: Longitudinal chloride profiles adjacent to tendons in Bay 5, Beam 17/4
(Total chloride content)

5.3.9 Grout chloride content
High levels of calcium and sulphate were observed for the grout. This is perhaps consistent with the finding that the grout contained Calcium Sulphate (or gypsum). The additional quantities of calcium and the presence of potassium may indicate that other cementitious products were also present.

<table>
<thead>
<tr>
<th>Table 5.8: Average concentration of ionic species determine for the grout</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>(a) Cationic Species</strong></td>
</tr>
<tr>
<td>Cations</td>
</tr>
<tr>
<td>% (grout)</td>
</tr>
<tr>
<td><strong>(b) Anionic Species</strong></td>
</tr>
<tr>
<td>Anions</td>
</tr>
<tr>
<td>% (grout)</td>
</tr>
</tbody>
</table>

### 5.3.10 Summary

It was found that chloride levels remained elevated with depth for each beam, most likely due to the addition of calcium chloride. No clear trends in chloride levels were observed in relation to the geometric location or orientation of the beams or between Faces C and D. The majority of chloride levels were found to be greater than 0.4% $Cl^-$ (cement), which is above the recommended chloride limit for prestressed and reinforced concrete. On average, Beam 17/4 had the highest chloride levels, and Beam 17/3 the lowest. The maximum chloride level approaching 5% $Cl^-$ (cement) was found at the surface level on Beam 17/4. Low chloride concentrations were found in the grout. Chloride profiles determined along the prestressing tendons showed elevated levels (mostly ranging between 0.4 and 1.0% $Cl^-$ cement) but there is some evidence to suggest that higher chloride levels existed in the web than the lower flange, suggesting that the lower flange perhaps protected the lower tendon from further chloride ingress. However, chloride concentrations averaged 1% $Cl^-$ (cement) for profiles determined at the beam soffits.
5.4 Carbonation Profiles

5.4.1 Introduction

In addition to chloride induced corrosion, the effects of carbonation are thought to be a primary mechanism responsible for deteriorating reinforced concrete structures. As discussed briefly in Chapter 3, carbonation is the name given to the phenomenon whereby carbon dioxide permeates the porous concrete and chemically alters the concrete it comes into contact with. Whilst carbonation does not cause deterioration of the concrete directly, it does pose a threat to the steel encapsulated in the concrete. The process of carbonation reduces the pH of the surrounding concrete to drop below 9.0. If steel is within the vicinity of carbonated concrete, the reduction in pH will cause the passive oxide layer around the steel to break down and corrosion may initiate.

Traditionally, structures most at risk of carbonation are those located in sheltered, humid environments or where carbon dioxide levels are high (for example, inner-city structures or internal faces of concrete buildings). Marine structures are generally thought to be most at risk from the aggressive chloride-laden marine environment rather than from carbonation effects. However, previous DIER inspections had identified carbonation depths of approximately 20mm which suggested that carbonation effects may be more influential than first thought.

The following section presents results from detailed investigations conducted on freshly cut beam faces and cores. Firstly, the mechanisms of and factors influencing carbonation are reviewed. Typical carbonation levels for coastal structures found in the literature are also highlighted. Then results are discussed.

5.4.2 Carbonation Mechanisms

The mechanism of carbonation has been reviewed thoroughly in the literature by Parrott [224] and Richardson [240–242], and a brief summary is now presented.

The process of carbonation commences with the presence of carbon dioxide at the surface of the concrete. Carbon dioxide occurs naturally in the atmosphere in concentrations of approximately 360 parts per million (ppm). This gas will dissolved into moisture
5.4 Carbonation Profiles

present at the surface or within the capillary pores of the concrete to form carbonic acid, dissociating into hydrogen and carbonate ions as shown in Equations 5.3 and 5.4

\[ H_2O + CO_2 \rightleftharpoons HCO_3^- + H^+ \] (5.3)

\[ HCO_3^- \rightleftharpoons H^+ + CO_3^{-2} \] (5.4)

At the same time, the solid cement hydration product calcium hydroxide, Ca(OH)$_2$, and other hydroxide constituents contained within the cement paste will dissociate in the pore water. This provides a highly alkaline solution that has a pH of greater than 12.5, depending on the quantity of sodium hydroxide and potassium hydroxide present [6, 152, 228]. The preferential reaction of the dissociated calcium hydroxide with the available carbonate ions results in the precipitation of insoluble calcium carbonate (CaCO$_3$), as shown in Equation 5.5.

\[ Ca(OH)_2 + 2H^+ + CO_3^{-2} \rightarrow CaCO_3 + 2H_2O \] (5.5)

Additional calcium hydroxide becomes available during the carbonation process with the breakdown of another hydration product, Calcium Silicate Hydrate (C-S-H) gel, which is only stable in the presence of calcium hydroxide [170, 210]. This reaction will continue within this vicinity until all the hydroxides are consumed, at which time the reaction front will advance further into the concrete; this is defined as the carbonation front, and the consumed hydroxide layer is now known to be carbonated. The progression of the front will be fuelled by the drawing in of more air containing carbon dioxide in order to establish concentration equilibrium between the carbonated and non-carbonated layers; this process is known as diffusion (Section 5.3).

As stated already, the physical ramifications for the concrete are not significant. In fact, carbonation of concrete may lead to a strengthening of the concrete and a reduction in its permeability [57, 163]. This is due to the creation of a residual network of silica, alumina and iron oxides which are remnants from the release of hydroxides from the carbonation-altered hydration products. This network is subsequently filled with the calcium carbonate precipitate. With the consumption of the hydroxide constituents, the pH of the concrete in this vicinity is reduced to approximately 9, with values ranging between
5.4 Carbonation Profiles

8 to 10 stated in the literature [55, 57, 240, 270, 282].

As has been previously discussed in Chapter 3, the passive oxide layer protecting the embedded reinforcement will breakdown when the pH of the surrounding concrete drops below 9. Therefore when the carbonation front reaches the depth of embedded steel, the process will gradually reduce the pH of the concrete, destroying the passive film and allowing corrosion to initiate. This process is inevitable, however the rate at which carbonation occurs will depend on a number of factors. This is discussed in Section 5.4.3.

5.4.3 The Rate of Carbonation

In order to determine the serviceability of a structure subjected to carbonation, engineers are interested in the rate of which carbonation will proceed and at what point the carbonation front will reach the reinforcing steel. Initially, the reaction at the surface of freshly set concrete is rapid due to the abundant supply of air, surface moisture and un-reacted calcium hydroxide, however this will slow once this layer has been consumed. Under “normal” circumstances and in comparison to chloride-induced corrosion, the rate of carbonation in concrete is quite slow; this is due to the slow diffusion process of the carbon dioxide into the pore water, which is approximately four orders of magnitude slower than the diffusion of carbon dioxide into air [57, 167]. Comparative rates between carbonation and chloride-induced corrosion have been discussed in Chapter 3.

The rate of carbonation can be influenced by a number of parameters. The most influential factors include the water/cement ratio, relative humidity, temperature and the concentration of carbon dioxide present in atmosphere [57, 167, 224, 242, 285]. These factors and their influences included:

- **Water/Cement Ratio:** reducing the w/c ratio reduces the porosity of the concrete. This in turn lowers the diffusion rate of carbon dioxide through the concrete and subsequently reduces the rate of carbonation.

- **Relative Humidity of the Concrete:** the relative humidity will regulate the degree of saturation in the pores. For a maximum carbonation rate, the optimum relative humidity is between 50 and 70% [224, 242, 285]. If the concrete is too dry or is saturated, the rate of carbonation will become negligible.
5.4 Carbonation Profiles

- **Air Temperature**: increasing the air temperature will result in an increased rate of carbonation, as long as there is pore water present (i.e. elevated temperatures have not promoted evaporation)

- **Concentration of Carbon Dioxide**: increasing the carbon dioxide concentration in air will increase the rate of carbonation. As shown in Figure 5.26 this may be a future concern due to a recorded increase in levels of atmospheric carbon dioxide [242, 323].

![Relative Carbonation Rate](image)

Figure 5.26: Relative carbonation rate with respect to environmental location [242]

- **Cement type**: the carbonation process depends on the presence of Calcium Oxide (CaO) in the cement; therefore a cement with a higher ratio of Calcium Oxide will result in a reduced rate of carbonation

- **Aggregate**: the orientation and diffusion characteristics of the large aggregate can influence the rate of carbonation. The low diffusivity of hard aggregate can prevent the progression of the carbonation front at this location, and as such carbonation will be irregular

- **Concrete Properties**: the permeability characteristics and the type of aggregate and cement can influence the rate of carbonation
5.4 Carbonation Profiles

- **Cracking:** with the event of cracking, new areas of non-carbonated concrete are exposed which can thus speed up the carbonation process. The progression of this new carbonation front will depend on the crack width and depth.

- **Orientation/Sheltering:** a concrete structure that is sheltered from rain will result in higher carbonation rates than one that is not. Internal concrete walls (such as an office building) will result in the highest rates of carbonation.

As previously mentioned in Chapter 2, a carbonation depth of 19mm on beams from the Sorell Causeway Bridge was recorded by Fa¸ cade Engineering in 1994. However, with reference to Figure 5.26 and the relevant literature [57, 167, 224, 242, 285], there are no guidelines to define whether this carbonation depth is excessive for concrete structures located within a marine environment. The literature recommends the use of an approximate parabolic formula which enables the forecasting of carbonation depths over time. This relationship is represented by Equation 5.6.

\[
d = K \cdot \sqrt{t} \tag{5.6}
\]

where:
- \(d\) = the carbonation depth (mm)
- \(t\) = time in years
- \(K\) = a constant

The constant \(K\) in Equation 5.6 is a function of the concrete properties and environmental conditions. For well compacted, low porosity concrete, \(K\) can be less than 2; where concrete is highly permeable and has low cement contents, \(K\) can be greater than 9. An overview of this relationship is shown in Figure 5.27. Based on this graph and using the DIER measured carbonation depth of 19mm, it is estimated that \(K\) may lie between 3 and 4. To further qualify these findings, some examples of carbonation depths measured from real-life coastal structures is shown in Table 5.9. This table also includes an estimate of the \(K\) constant for comparison. It should be noted that these values are average carbonation depths and do not appear to recognise the presence of cracks or climatic influences.
5.4 Carbonation Profiles

Figure 5.27: Depth of carbonation over time, approximated by $d = K \sqrt{t}$

Table 5.9: Examples of carbonation depths from the literature regarding real-life coastal structures

<table>
<thead>
<tr>
<th>Reference</th>
<th>Location</th>
<th>Carbonation Depth</th>
<th>Time</th>
<th>K Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broomfield [67]</td>
<td>unknown; poor concrete</td>
<td>16mm</td>
<td>16 years</td>
<td>≈ 4</td>
</tr>
<tr>
<td></td>
<td>unknown; good concrete</td>
<td>4mm</td>
<td>20 years</td>
<td>&lt; 1</td>
</tr>
<tr>
<td>Bruce et al. [71]</td>
<td>New Zealand</td>
<td>13mm (inside face)</td>
<td>42 years</td>
<td>2-3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7mm (outside face)</td>
<td>42 years</td>
<td>1-2</td>
</tr>
<tr>
<td>Seki [224]</td>
<td>Japan</td>
<td>60mm</td>
<td>15 years</td>
<td>≈ 15</td>
</tr>
<tr>
<td>Unknown [224]</td>
<td>Netherlands</td>
<td>&lt; 5mm</td>
<td>3-62 years</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>Mackechnie [177]</td>
<td>South Africa</td>
<td>10mm</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Poupard et al. [226]</td>
<td>France</td>
<td>1mm</td>
<td>40 years</td>
<td>&lt; 1</td>
</tr>
</tbody>
</table>

5.4.4 Equipment and Methodology Adopted

Equipment and methodology procedures are discussed in Appendix B. In summary, carbonation depths were determined by using a phenolphthalein indicator, which highlighted zones of carbonated concrete by means of a colour change (pink = non-carbonated concrete, clear = carbonated). The boundary between the colour change occurs at a pH of approximately 9.5, however this will underestimate the start of carbonation front, which is identified by pH zones of 11.5 [91, 167, 224, 242]. However as corrosion of reinforcement initiates at a pH of 9, the phenolphthalein indicator method was deemed to be sufficient.
5.4 Carbonation Profiles

Samples tested included cores taken from selected diaphragms and freshly cut beam faces from Bay 5 for each beam. The indicator solution was sprayed onto the surface of the samples and the extent and depths of carbonated concrete were subsequently measured.

5.4.5 Results of Carbonation Depth Measurement

Carbonation depths were found to agree with the greater estimates made by Façade Technology [118] rather than the low estimates made by Taywoods [278]. For beam cross-sections from Bay 5, carbonation depths varied depending on its location across and along the beam. Greater levels were found adjacent to Diaphragms (iv) and (v), with lower levels recorded towards the centre of the bay. Greater depths of carbonation were typically found across the webs and towards the upper flange, with minimal carbonation recorded across the lower flange. Average carbonation depths along the webs ranged between 5 and 30mm. Depths along the lower web were typically less than 5mm. Instances of cracking and spalling were associated with deeper carbonation, with the full width of the web for Beams 17/3 and 118 found to be fully carbonated where longitudinal cracking was worst. Some carbonation was observed across grout/concrete interfaces where voids were present in tendons; however, most observations were superficial and the remainder of the grout was still highly alkaline exhibiting no evidence of neutralisation around prestressing strands (with the exception of the aforementioned case). The following sections detail the findings for each beam.

5.4.5.1 Beam 17/4

Zones of carbonation were predominantly confined to areas closest to the two diaphragms of Bay 5, which is shown in Figure 5.28. Face C was notably more carbonated than Face D, especially at sites which were marked by spalling and rust staining of the shear ligatures. Here, carbonation depths were typically between 10 and 20mm. As noted previously in Chapter 4, the cover to the shear ligatures is less than 20mm. Whether the carbonation front had reached the reinforcement first, or spalling/cracking initiated prior to carbonation is unknown. However it is clear that the concrete is no longer affording protection to the concrete through its alkalinity. Figure 5.29 shows an example of carbonated concrete overlaying a corroding shear ligature.
5.4 Carbonation Profiles

The deepest carbonation was observed at the corner junction between the web and upper flange, with depths approaching 30mm. Minimal, if any, carbonation was noted across the base flange and the carbonation front had not approached the tendons.

5.4.5.2 Beam 17/3

Carbonation depths across bay 5 were found to be the most significant of all three beams. The webs of Face D exhibited deeper carbonation than Face C; carbonation zones closest to each diaphragm were also deeper (Figure 5.30). Carbonation depths at the webs were more consistent, averaging between 10 and 30mm (with the exception of Section C, which had significantly lower carbonation depths across the upper regions of the web). Car-
5.4 Carbonation Profiles

Figure 5.29: Carbonated concrete in the vicinity of spalling adjacent to Diaphragm (iv), Beam 17/4

Carbonation was not as pronounced at the web/upper flange junction as observed on Beam 17/4, however, a greater proportion of the upper flange soffit was carbonated to depths between 5 and 20mm.

The deepest carbonation was observed adjacent to the lower tendon for almost the full length of Bay 5. Sections C and D showed depths of carbonation to almost the full width of the web. Section B exhibited similar patterns, although not to the same extent. Longitudinal web cracking observed in the vicinity appears to have facilitated the deeper carbonation. Carbonation appears to have followed similar cracking observed adjacent to the upper tendon appears in Section D, but not on Section C. In fact, there was no observed carbonation noted adjacent to this cracking on Face C of Section C, yet less than 200mm from this point the web is completely carbonated adjacent to the lower tendon. Figure 5.31 shows photographic evidence of the carbonation profile from Section C.

As noted for Beam 17/4, lower carbonation depths were observed across the soffits of the lower flanges which averaged less than 5mm collectively. This result was confirmed by observations made on several cores (Figure 5.32a). Typical carbonation profiles of the lower flange are shown in Figure 5.32b. The colour change of the indicator solution was not as intense at the lower flange compared to the other tests. However, a slight colour
change was noted between the upper and lower regions of the flange, indicating that the region adjacent to the lower tendon was more carbonated than the soffit. A strong colour response was observed in the grout, indicating its high alkalinity. The longitudinal reinforcement was found to be in good condition compared with the prestressing strands which had undergone some section loss.
5.4 Carbonation Profiles

Figure 5.31: Carbonation profile of Section C from Bay 5, Beam 17/3 (see Figure 5.30)

(a) Carbonation profile of lower flange, Beam 17/3  (b) Low carbonation levels along beam sofit

Figure 5.32: Examples of low carbonation depths on the base flange

5.4.5.3 Beam 118

Carbonation patterns observed across Bay 5 of Beam 118 were similar to those found for Beam 17/3 (Figure 5.33). Carbonation depths were not as deep as for Beam 17/3, especially adjacent to the lower tendon and longitudinal cracking. Depths averaged between 10 and 20mm. Data was not available for Face D due to the large amount of spalling
which had disrupted the accuracy of the phenolphthalein spray. Carbonation did not appear to follow the cracking noted adjacent to the upper tendon on Section D.

Figure 5.33: Carbonation profiles for Bay 5, Beam 118
The carbonation front was significantly influenced by the presence of aggregate. It was noted for all cross-sections that there was a predominance of well-compacted aggregate throughout the centre of the section, with aggregate sizes being large and elongated. Towards the surface, the frequency of the aggregate dropped, leaving zones of cement susceptible to carbonation. Figure 5.34 shows an example where a lack of aggregate has resulted in deeper carbonation adjacent to the upper tendon. As for Beam 17/4 and 17/3, very little carbonation had occurred across the base flange.

Figure 5.34: Carbonation profile of web from Bay 5, Beam 118

5.4.5.4 Additional carbonation observations

A number of other observations on the extent of carbonation were made during the course of this testing. These are discussed in the following paragraphs.

Grout Voids

As noted for the transverse tendons in Chapter 4, small voids were observed in the grout at the tops of both upper and lower longitudinal tendons. At these locations, small levels of carbonation were observed, as shown by representative samples in Figure 5.35. The carbonation was not very deep, and the majority of the grout retained its high alkalinity, with pH levels in this vicinity measuring between 12 and 13.5. The carbonated grout
averaged a pH of 9. The low levels of carbonation may have been due to the limited supply of free carbon dioxide within the tendons.

Figure 5.35: Examples of grout carbonation at void locations

Figure 5.36 shows an example where a grout void was once present but had subsequently filled with corrosion product. The colour change observed in Figure 5.36 shows an area with a significantly reduced pH adjacent to the prestressing strands. The pH in this zone (using pH indicator sticks) was measured to range between approximately 5 - 7. It is thought that this observation is associated with the occurrence of “chloride weeping”, which is discussed in more detail in Chapter 6.
5.4 Carbonation Profiles

**Isolated, deep carbonation**

Pockets of isolated and deep carbonation were observed on several cores retrieved from the beams, as shown in Figure 5.37. For both of the examples shown, the cores were retrieved from the web of Bay 4 of Beams 17/3 and 118. Beam 17/3 showed an isolated “wedge” of carbonation (Figure 5.37a), however Beam 118 shows more significant carbonation (Figure 5.37b). For this latter example, the carbonation front extended to the outer tendon perimeter of the upper tendon, but it did not appear that the adjacent grout had become carbonated, as shown by the strong colour reaction. The strand at this location remained in good condition, however small, isolated pits were observed that had initiated on the steel. These instances appeared to be random and no explanation can be offered as to why this may be the case.

![Beam 17/3](image1.png)  ![Beam 118](image2.png)

Figure 5.37: Examples of unusual, deep carbonation in core samples

**Advancement of carbonation front**

The progression of the carbonation front was not always consistent, often being influenced by the location of aggregate (as previously discussed, Figure 5.34). However, the carbonation front was in some cases found to be at different stages, seemingly unaffected by the presence of aggregate (Figure 5.38). The reason for this pattern is unknown.
5.5 Additional Concrete Properties

5.4.6 Summary

It has been shown that average carbonation depths between 20 and 30mm were found on the webs and the soffit of the upper flange for all beams. Deeper carbonation was found on zones adjacent to Diaphragms (iv) and (v). In most cases, the carbonation front had reached or was in close proximity to reinforcement. The most significant levels were measured adjacent to longitudinal web crack zones on Beams 17/3 and 118, where the carbonation front has progressed through the full width of the web at the lower tendon location. It is uncertain whether the levels of carbonation observed for these tests are considered to be excessive for a coastal structure, however based on similar case studies it appears that this is the case (see Chapter 3). These results will be discussed in relation to steel condition and chloride levels with respect to chloride binding in Chapter 7.

5.5 Additional Concrete Properties

Previous mention has been made regarding the role that concrete properties play in the corrosion process for reinforced and prestressed concrete. In particular, the permeability characteristics and cement/binder contents are considered to be most crucial. Several of these properties were examined in the present study. The majority of information was obtained in consultation with the University of Canterbury, New Zealand, as well as reviewing information from previous inspection reports (such as the 1994 Façade and the 1997 Taywood Engineering reports [118, 277, 278]). Other tests included the determination of the apparent volume of permeable voids of concrete cores in accordance with
5.5 Additional Concrete Properties

AS1012.21 [34]. A brief discussion of these examinations follows, with full consultancy reports and relevant literature reviews are found in Appendix C.

5.5.1 Cement and Binder Content

The cement content of the beams from the Sorell Causeway Bridge has been previously determined in the reports conducted by Façade Engineering in 1994 [118] and Taywood Engineering in 1997 [277, 278]. In these reports, test results found that the cement content of the precast I beams ranged between 15.1 and 22.3% by mass of concrete.

Unfortunately, no specifications have been found in relation to the amount of binder, or calcium chloride, added to the beams at the time of casting. However, based on historical literature it is thought that these levels may range between 0.5 and 2% by weight of cement [13, 204, 230].

5.5.2 Permeability Characteristics of the Concrete

Concrete permeability characteristics can influence the rate at which water, oxygen, carbon dioxide and chlorides can ingress through the concrete to the embedded reinforcement (see Chapters 3 and 4). Therefore several tests have been conducted to gain insight into these characteristics, with results discussed in the following paragraphs.

Oxygen Permeability Test

This test was conducted in order to determine the gas permeation properties of the concrete, which influences the ease at which oxygen and carbon dioxide can permeate to the steel. Table 5.10 shows the relationship between the Oxygen Permeability Index (OPI) and the durability characteristics of the concrete.

Table 5.10: Suggested durability classifications in relation to the oxygen permeability index [8]

<table>
<thead>
<tr>
<th>Oxygen Permeability Index (log scale)</th>
<th>Durability Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 10</td>
<td>Excellent</td>
</tr>
<tr>
<td>9.5 - 10</td>
<td>Good</td>
</tr>
<tr>
<td>9.0 - 9.5</td>
<td>Poor</td>
</tr>
<tr>
<td>&lt; 9.0</td>
<td>Very poor</td>
</tr>
</tbody>
</table>
Full results for this test can be found in Appendix C. In summary, the OPI for Beams 17/3 and 118 is 10.75 and 9.5, which relates to good and poor oxygen permeability characteristics respectively (i.e. Beam 118 is more susceptible to gas permeation than Beam 17/3). Oxygen permeability results were not obtained for Beam 17/4.

**Sorptivity Test**

In the sorptivity test, the amount of water that is absorbed unidirectional by the concrete. Table 5.11 provides recommended limits for sorptivity results in relation to concrete durability. Specific results for this test can be found in Appendix C, however in summary the sorptivity values for Beams 17/3 and 118 are 3.6 and 6.27 respectively, indicating that both beams had reasonable durability characteristics, but Beam 118 had greater porosity. Sorptivity results were not obtained for Beam 17/4.

<table>
<thead>
<tr>
<th>Sorptivity ( \text{mm}/\sqrt{\text{hr}} )</th>
<th>Durability Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 6</td>
<td>Excellent</td>
</tr>
<tr>
<td>6 - 10</td>
<td>Good</td>
</tr>
<tr>
<td>10 - 15</td>
<td>Poor</td>
</tr>
<tr>
<td>&gt; 15</td>
<td>Very poor</td>
</tr>
</tbody>
</table>

**Apparent Volume of Permeable Voids**

As an additional method, concrete core samples were tested in accordance with AS1012.21 [34] in order to determine the permeability characteristics of the concrete. Specifically, the immersed and boiled absorption levels and the apparent volume of permeable voids (AVPV) were calculated. A summary of these results are found in Appendix C. In summary, the average AVPV for Beams 17/4, 17/3 and 118 were found to be 9.3, 7.2, and 13% (by mass of concrete) respectively. This indicates that Beam 17/3 had the least amount of voids and thus lower permeability of the three beams; Beam 118 was the opposite.
Chapter 6

Corrosion Observations

6.1 Introduction

It has been noted previously that the corrosion of steel in concrete presents a threat to the service life of reinforced or prestressed concrete. This is mainly due to:

- Loss of concrete section (spalling)
- Loss of ductility
- Loss of bond (loss of frictional characteristics)
- Loss of cross-sectional area of the steel

This chapter will describe the physical condition of selected steel sections retrieved from the three test beams of the Sorell Causeway Bridge. Section 6.2 will examine the distribution and general characteristics of corrosion observed. Section 6.3 will consider the corrosion products encountered. Some additional observations are made in Section 6.4.

6.2 Steel Condition

Prestressing strands and conventional reinforcement from Bay 5 of each beam were inspected for the extent and severity of corrosion. In particular, cross-sectional area losses were determined along steel lengths for comparison with findings described in Chapters 4 and 5. The following sections detail the condition of the steel, including discussions on
6.2 Steel Condition

the estimation of section losses and the observed profiles with respect to its embedded location. Corrosion quantification results are summarised graphically, with particular focus on the prestressing strands.

6.2.1 Background and Methodology Adopted

Assessing the condition of the steel and quantifying subsequent steel section losses can be carried out through a variety of means. The predominant methods commonly used in practice are either gravimetric weight loss or by measuring the reduction in the diameter of the steel. The latter technique was employed for the current investigation, with an emphasis on pitting corrosion due to the chloride contamination of the concrete. The standard ASTM G46-94 [45] was used as a basis for evaluation.

For each beam, selected concrete sections were broken open one at a time using a sledge hammer to minimise damage to the encased steel. Each steel segment was allocated a unique number for identification. The location and orientation for each steel specimen was noted. Subsequently, the steel was subjected to detailed visual inspection, which included the documentation of section losses and pitting. This included photographing the steel bars against a scale so as to ensure a complete visual record. Steel sections were then carefully cleaned of corrosion products using a stiff wire brush in accordance with ASTM G1-03 [44]. This section will focus predominantly on the steel section losses observed. For discussions pertaining to corrosion products and other observations see Sections 6.3 and 6.4.

The quantification of steel section losses was carried out by measuring steel bar diameters using a digital vernier calliper at several preset intervals per 100mm length along the bar, which were averaged per length to determine the overall cross-sectional area reduction. Cross-sectional areas were calculated based on the simplified area reduction model shown in Figure 6.1a [107, 293]. The model was generally consistent with the pitting morphology observed in the present work (Figure 6.1b gives an example).

It was assumed that the depth of the pit concavity was the same as the measured decrease to the upper edge of the pit, with an area upper limit set to 50% (Figure 6.1c). Where areas greater than 50% are registered, the pit morphology was assumed to change to a
flat, semi-circular profile (also noted from physical observations), and the remaining steel area was calculated based on the properties of a simple segment (Figure 6.1d).

![Diagram](image1)

(a) A schematic of section losses due to pitting (strands only)
(b) A typical example of a strand cross-section with localised pitting

![Diagram](image2)

(c) Model for where cross-sectional area is 50% or less
(d) Model for where cross-sectional area is greater than 50%

Figure 6.1: Adopted model for assessing steel cross-sectional area losses
In order to assess the extent and severity of corrosion on a holistic basis, all steel section loss data has been represented graphically and relevant diagrams found in Sections 6.2.2 and 6.2.3. These diagrams represent the average steel condition per 100mm length; it does not highlight specific instances of pitting. A condition classification system was established for the present work to categorize the severity of the steel condition. No specific standard exists for such classification systems, therefore similar case studies have been utilised. Many of these studies have systems that are based purely on the visual appearance of the steel; for example, McCann and Forde [184] provide an assessment of steel condition based on “corroded” versus “non-corroded” regions, although it is unclear what visually defines these regions. Qian et al. provide similar corrosion benchmarks in their case study on repaired concrete bridge slabs [231]. Whilst this approach appears sensible, it is subjective.

Other studies relate steel condition to quantifiable average section losses. Palsson and Mirza [222] recommend four corrosion levels based on average cross-sectional area losses, with Group 1 representing area losses of less than 10% and losses greater than 30% equate to Group 4. Woodward and Williams [321] suggest five levels, ranging from “no corrosion” (assumed to be no sectional losses) to severe losses greater than 50%. Poupard et al [226] use a combination of visual observations and calculated average losses to determine corrosion severity.

Therefore in keeping with the above reviewed systems, a corrosion severity classification system was developed for the assessment of the conventional reinforcement and prestressing steel condition retrieved for the current work. Two separate systems were devised for the conventional reinforcement and prestressing steel due to the size and structural significance of the steel. The classification of the severity of corrosion observed for both systems is centred around the corresponding cross-sectional area losses observed and measured on the steel. For the conventional reinforcement, three corrosion severity levels were established of which is described in Table 6.1 and shown visually in Figure 6.2.
6.2 Steel Condition

Table 6.1: Steel condition classification system for conventional reinforcement

<table>
<thead>
<tr>
<th>Corrosion Level</th>
<th>Description</th>
<th>Cross-sectional Area Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excellent condition; no rust stains</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Rust stains; some section loss</td>
<td>≤ 25%</td>
</tr>
<tr>
<td>3</td>
<td>Active corrosion; significant section losses</td>
<td>&gt; 25%</td>
</tr>
</tbody>
</table>

Figure 6.2: Examples of corrosion levels adopted for conventional reinforcement

To assess the condition of the prestressing strands, the steel was grouped into four representative groups relating to the location of the strands within the duct, namely upper, lower and web-facing strands (Figure 6.3). To determine the average condition of the strand group, cross-sectional area losses per 100mm length of individual strands were collated and averaged in accordance with its position within the duct. As for the conventional reinforcement, a classification system based on the severity of corrosion was established and is summarised in Table 6.2, with visual representations shown in Figure 6.4.

To allow the collation and comparison of test results with the physical condition of the steel, graphical representations of steel condition data were created for both conventional reinforcement and prestressing steel. These show the condition of the steel in relation to its position within the beam and have been colour coded for ease of reference. The following
6.2 Steel Condition

Figure 6.3: Strand grouping for graphical corrosion condition representation

Table 6.2: Steel condition classification system for prestressing steel

<table>
<thead>
<tr>
<th>Corrosion Level</th>
<th>Description</th>
<th>Cross-sectional Area Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excellent condition; no rust stains</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Rust stains; no section loss</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Onset of corrosion; ≤ 10% corrosion</td>
<td>≤ 10%</td>
</tr>
<tr>
<td>4</td>
<td>Moderate corrosion; 10 - 25% corrosion</td>
<td>10 - 25%</td>
</tr>
<tr>
<td>5</td>
<td>Significant corrosion; 25 - 50% corrosion</td>
<td>25 - 50%</td>
</tr>
<tr>
<td>6</td>
<td>Severe corrosion; ≥ 50% corrosion</td>
<td>≥ 50%</td>
</tr>
</tbody>
</table>

Figure 6.4: Examples of corrosion levels adopted for prestressing steel
sections discuss the visual condition observations made for the conventional reinforcement and prestressing steel retrieved from Bay 5 of each beam with respect to the corrosion classification systems established. General corrosion observations and conditions are also discussed.

6.2.2 Conventional Reinforcement

6.2.2.1 Summary of observed conditions

Corrosion was found to be most pronounced in locations where spalling and cracking had occurred. In these locations, cross sectional area losses were substantial with area reductions often greater than 50%. In some cases losses were closer to 100% of the cross-section, which had most likely been in this condition whilst the bridge was in service. Zones of corrosion varied between the three beams. Beam 118 had a greater number of bars that exhibited corrosion initiation compared to Beams 17/3 and 17/4, although similar degrees of steel section losses had occurred in zones of cracking or spalling. Corrosion appeared to occur preferentially. Face C of Beams 17/4 and 17/3 or Face D of Beam 118 had higher incidences of corrosion when compared to the opposite face. In many cases, significant corrosion and subsequent section loss was found to occur only 100mm away from what appeared to be a completely uncorroded section of bar, with only minimal amount of corrosion product evident. Such localised instances were generally found to occur within crack zones. The following paragraphs will detail key observations made for each beam.

Beam 17/4

Generally, instances of corrosion on conventional reinforcement from Bay 5 coincided with observed spalling. As can be seen in Figure 6.5, only the shear ligatures presented cases of significant corrosion and section loss, with the lower longitudinal reinforcement found to be in excellent condition and no evidence of corrosion. Section losses were most significant and substantial on the three ligatures retrieved from Face C that were closest to Diaphragms (iv) and (v), where the majority of spalling and rust staining was observed. In comparison, little or no corrosion was observed on ligatures from Face D or on those located mid-bay.
6.2 Steel Condition

Figure 6.5: Condition of conventional reinforcement across Bay 5, Beam 17/4
6.2 Steel Condition

Figure 6.6a shows a photograph of the upper half of this section, showing the exposed shear ligatures in situ on Face C. The ligatures at Points A and C showed significant corrosion along the full length of the web, with consistent section losses. The ligature at Point B showed similar corrosion along the upper and lower parts of the bar (which appear to correspond to the post-tensioning tendons), however a section of bar essentially unmarked by corrosion existed in between these zones. Figure 6.6b shows the base flange of the section adjacent to the one shown in Figure 6.6a. Corrosion was observed on ligatures at Points A and B, however section losses were relatively minor. No spalling or cracking was evident here. Corrosion appeared to have stemmed from adjacent strands from the lower tendon rather than the shear ligatures (see Point C). This is discussed further in Section 6.2.3.

![Figure 6.6: Corrosion on shear ligatures from Bay 5 adjacent to Diaphragm (v)](image)

Section losses ranged between 50 to 75% in places where corrosion was most severe. Steel profiles affected by corrosion were predominantly irregular and jagged, and steel surfaces were friable in places. Figure 6.7 shows representative samples of shear ligatures retrieved from spalling zones. Figure 6.7a shows the irregular surface often encountered; Point A highlights the flattening edge towards the top of the ligature. This latter observation was noted on most ligatures, with the flat edge facing outwards towards the concrete surface. Severe localised section losses were observed at Points B, A and A & B of Figures 6.7a, 6.7b and 6.7c respectively. These localised areas show elongated pits, which had brittle edges and friable corrosion layers.
Figure 6.7: Examples of corrosion profiles observed on reinforcement from Beam 17/4

Figure 6.7d shows a combination of the observations above, with a localised circular pit at the centre of the ligature. An even, elliptical edge is highlighted by Point B, marking the boundary of the section loss for this ligature. No rust staining was observed adjacent to this area. Similar observations were made on ligatures shown in Figure 6.8. These isolated pits, circular/elongated in morphology, were observed to have regular, smooth
edges and section losses of between 1-2mm. Again, very little corrosion product was noted adjacent to these pits. The base of the pits were smooth and free of bulk corrosion products. Figure 6.8b was observed to have a black, powdery base with a bright metallic centre. These observations are discussed further in Section 6.4.

![Figure 6.8: Examples of isolated pits on reinforcement from Beam 17/4](image)

(a) Distinct, circular pit

(b) Elongated pit with a bright, metallic base

Beam 17/3

As anticipated, severe corrosion was found on shear ligatures and lower longitudinal reinforcement in web crack zones and towards the base of the beam, although activity extended a small way beyond the crack surfaces. Severity of corrosion increased closer to Diaphragm (iv), although the most significant steel section losses were recorded approximately 700mm from this diaphragm (within the crack zone). As observed for Beam 17/4, reinforcement located along Face C showed more significant corrosion than the opposite face. The distribution of corrosion and subsequent section losses is shown in Figure 6.9. Instances of rust staining on the surface of the concrete generally coincided with the location of shear ligatures or the longitudinal reinforcement along the beam soffit. However, where severe corrosion had occurred on the longitudinal bars, there was either little or no corrosion on the soffit of adjacent shear ligatures.
6.2 Steel Condition

Longitudinal rebars retrieved from the base of the beams had suffered significant section losses running in parallel with the beam soffit. Losses ranged between 50 and 100% of the cross-sectional area. Figure 6.10 shows representative surfaces of longitudinal reinforcement retrieved from the most corroded zones. As for Beam 17/4, the surfaces were irregular, pitted and friable. Flat edges were observed (Point A, Figure 6.10a), which were marked by deep, localised pitting (Point B, Figure 6.10a). In some instances, the pit surfaces were more concave in nature (Figure 6.10c).

Within some corrosion zones, brittle corrosion surfaces were observed with cracks running approximately perpendicular to the bar axis. Examples are shown in Figure 6.11. In some cases, the cracks were obscured by corrosion products (Figure 6.11a), and in other cases cracks were clearly visible and had a brittle appearance (Figure 6.11b).
6.2 Steel Condition

Figure 6.9: Condition of conventional reinforcement across Bay 5, Beam 17/3
6.2 Steel Condition

Figure 6.10: Examples of corrosion profiles observed on reinforcement from Beam 17/3

Figure 6.11: Examples of cracking on reinforcement within brittle corrosion products from Beam 17/3
Deep, isolated, circular pits were observed on some rebars with little or no section losses adjacent to these pits. Figure 6.12 shows some typical examples, which are characterised by a well-defined perimeter and pit depths of up to 50% of the bar cross-section. The remainder of the bar remained intact with minimal section loss. Little or no corrosion product were observed in the base of the pits (Figure 6.12b), which were usually smooth.

An unusual form of pitting was noted across a longitudinal rebar from Diaphragm (iv), as shown in Figure 6.13. Section losses of the bar were substantial and the pattern of pitting appeared to have occurred in rows or tunnels (Figure 6.13a). The surface was covered in a tightly adhering black “shimmery” rust (identified later as magnetite by X-Ray Diffraction (XRD)), which oxidised to dark brown rust within 24 hours of atmospheric exposure (Figure 6.13b). Observing the surface using Scanning Electron Microscopy (SEM) showed a series of channels that had undercut the bar perimeter (Figure 6.14). The pit surface itself was relatively smooth with limited corrosion product accumulating at the base of some channels. Microcracks were also evident in the channels.
6.2 Steel Condition

Figure 6.13: Unusual pitting observed on longitudinal rebar from Beam 17/3

Figure 6.14: SEM image of unusual pitted area of a longitudinal rebar from Beam 17/3 (see Figure 6.13)

Corrosion of the shear ligatures revealed findings similar to those discussed in the previous paragraphs. The most severe cross-sectional area losses (up to 100%) were found in areas adjacent to longitudinal cracking. Section losses were most significant on ligatures from Face C. Unusually, section losses were not as significant on shear ligatures directly adjacent to longitudinal rebars undergoing severe corrosion. Figure 6.15 shows an example of severe corrosion on a shear ligature retrieved from Face C close to Diaphragm (iv). The
ligatures had undergone severe localised corrosion, with complete section losses observed at the top of the ligature (Point C), and section losses greater than 50% at Points B and C. These points were located adjacent to the lower prestressing tendon. Figures 6.15b and 6.15c show similar examples. It is interesting to note that 100% loss of cross-section (Point A on both Figure 6.15b and 6.15c) co-existed adjacent to mostly-uncorroded condition bar (Point B on both Figure 6.15b and 6.15c).

![Figure 6.15: Examples of section losses on shear ligatures from Beam 17/3](image)

(a) Example #1

(b) Example #2

(c) Example #3

Deep, isolated pits were also found in some ligature sections as for the longitudinal rebars. Figure 6.16 shows an unusual case of corrosion along a shear ligature adjacent to cracking near Diaphragm (v). Severe corrosion had occurred adjacent to longitudinal crack zones, resulting in complete section losses at Point A. However, further along the ligature where overall section losses were not as significant, deep, concentric pits approximately 1mm deep marked the surface (Points B and C). As noted for Figure 6.15, these pits existed adjacent to a passive surface. Figure 6.17 shows a similar case, where a bar retrieved from Diaphragm (iv) exhibited a deep elongated pit directly adjacent to areas of minimal...
section loss (Point A). Complete section losses of the adjacent bar at Point B were also observed.

Figure 6.16: Isolated, concentric pits on shear ligature adjacent to longitudinal cracking, Beam 17/3

Figure 6.17: Severe section losses on steel from Diaphragm (iv), Beam 17/3

An example of tunnelling corrosion similar to that noted in Figure 6.14 was observed on the tip of a shear ligature retrieved from a longitudinal crack zone. Figure 6.18 shows two different perspectives of the same bar, showing that the outer surface of the bar was relatively intact in comparison to its centre. This ligature was retrieved “as-is” from the
concrete, indicating that the ligature had completely corroded whilst the bridge was in service. This was representative of a number of bars from all three beams.

![Figure 6.18](image)

Figure 6.18: “Tunnel-like” corrosion on tip of shear ligature from Beam 17/3

**Beam 118**

Corrosion observations made on conventional reinforcement from Beam 118 were very similar to findings made for Beam 17/3. Corrosion was noted to have initiated on most steel from Bay 5 (especially the shear ligatures), predominantly adjacent to spall and crack zones from Face D. The worst corrosion was observed in sections closest to Diaphragm (iv). The condition of rebars retrieved from Face D were worse than those from Face C. A greater percentage of bar area was affected by corrosion than previously seen on Beams 17/4 and 17/3; the distribution of observed corrosion is shown in Figure 6.19. From Figure 6.19, it can be observed that where severe corrosion had occurred on longitudinal rebars, the shear ligatures were also noted to be in poor condition (dissimilar to that noted for Beam 17/3).

Greater cross-sectional area losses were noted on bar sections facing outwards. Figure 6.20 shows typical examples of this phenomenon. Severe corrosion and a flattened edge were observed on the underside of longitudinal rebars retrieved from the beam soffit. Note the concave nature of the corrosion profile shown in Figure 6.20b. Edges were irregular, brittle and sharp. Isolated and deep pits were also observed, similar to those noted for Beam 17/3 (see Figure 6.12).
6.2 Steel Condition

Figure 6.19: Condition of conventional reinforcement across Bay 5, Beam 118

[Diagram showing the condition of steel with symbols indicating no corrosion, some corrosion, and severe corrosion with percentage indications.]
6.2 Steel Condition

Severe pitting and section losses were observed on shear ligatures in areas similar to those noted for the longitudinal rebars. Figure 6.21 shows representative samples of three sets of shear ligatures retrieved from the centre of Bay 5. Differences between the ligatures from Faces C and D were observed with respect to extent and severity. Irregular surfaces and complete section losses defined ligatures from Face D at a lower level than the opposing ligatures. Figure 6.22 shows a close-up photograph of a ligature in a similar location with significant pitting. Note the irregular and intricate detail observed along the edge of this pit, which is approximately 1-2mm deep. Undercutting of the pit edge had also occurred and a white residue was observed at the base of the pit. Figure 6.23 shows a typical example where isolated, deep pitting had occurred in conjunction with minimal corrosion product. A fine crack was observed to have emanated from the pit through the steel, as shown by Point A.

An unusual pit was observed on a rebar from the base of Diaphragm (iv) as shown in Figure 6.24, similar to that observed in Figure 6.13. Channels similar to the latter example were observed on this rebar, however edges were smoother and not as irregular. Corrosion patterns appeared to follow a flow-like pattern, with rims not dissimilar to a thumb-print. An older, oxidised section of this pitting (Point A) was observed adjacent to a freshly-exposed part of the pit (Point B), of which dark, green rusts were observed.
6.2 Steel Condition

Figure 6.21: Corrosion profile of shear ligatures retrieved from Bay 5, Beam 118

Figure 6.22: Deep and irregular pitting on shear ligature from Beam 118

Figure 6.23: Deep, isolated pitting on shear ligature from Beam 118
6.2 Steel Condition

(incidents of green rusts are discussed further in Section 6.3.4.2). Approximately 50% of the bar area had been lost due to corrosion at this location.

![Unusual pitting/section losses on rebar from Diaphragm (iv), Beam 118](image)

Figure 6.24: Unusual pitting/section losses on rebar from Diaphragm (iv), Beam 118

### 6.2.3 Prestressing Strands

### 6.2.4 Summary of General Observations for Each Beam

On average, Beam 17/3 exhibited the worst condition, followed by Beam 118 and then Beam 17/4. The graphical representation of the prestressing steel condition is shown in Figures 6.25, 6.26 and 6.27 for Beams 17/4, 17/3 and 118 respectively. Areas of active corrosion were observed where a predominance of spalling and cracking had occurred similar to that observed on ligatures in Section 6.2.2. The worst corrosion in Beams 17/3 and 118 were located approximately 700mm from Diaphragm (iv), where in some instances complete steel section losses were observed. It was noted that some strands from Beam 17/4 were also exhibiting corrosion however, no visible evidence of this corrosion was noticeable on the concrete surface (Figure 6.25).

As a general trend, more significant and extensive corrosion was observed on strands from the lower tendon rather than the upper tendon. Additionally, strands from the “top group” (Figure 6.3) exhibited the greatest cross-sectional area losses closely followed by strands adjacent to the web faces. Preferential corrosion between faces was also observed; for example, corrosion was more significant on strand groups recovered from Face C for Beams 17/4 and 17/3, whilst the opposite was true for Beam 118. The following sections provide an overview of corrosion observations made for the prestressing strands.
Figure 6.25: Condition of prestressing strands across Bay 5, Beam 17/4
Figure 6.26: Condition of prestressing strands across Bay 5, Beam 17/3

- **No corrosion; good condition**
- **Active corrosion; $A_s > 90\%$**
- **Significant corrosion; $50\% < A_s < 75\%$**
- **Rust staining; no section loss**
- **Moderate corrosion; $75\% < A_s < 90\%$**
- **Severe corrosion; $A_s < 50\%$**
Figure 6.27: Condition of prestressing strands across Bay 5, Beam 118
6.2.5 Strand profiles and corrosion characteristics

Overall, section losses on corrosion-affected strands resembled profiles similar to general corrosion. Section losses were more gradual and corrosion edges not as ragged as those observed on conventional reinforcement. However, a variety of corrosion profiles were observed and are now discussed.

As previously stated, the worst corrosion zones were observed adjacent to Diaphragm (iv). At these locations, strands from both the upper and lower tendons had severe cross-sectional area losses. In some cases complete losses were observed which had occurred whilst the bridge was still in service. Strand cross-sectional profiles with significant corrosion were reduced from a circular to an elliptical shape with a flat, planar upper edge extending along strand lengths for most of Bay 5; strands from areas with the worst corrosion had the most pronounced profile of this type. Representative samples are shown in Figure 6.28, with longitudinal profiles shown by Figure 6.28a and cross-sectional profiles shown in Figure 6.28b. An insitu example of the planar strand surface that has undergone severe corrosion is shown in Figure 6.29.

Section losses were not always consistent. In some locations, large variations in strand section losses would occur over short lengths along the bar. For example, section losses could vary from losses of 20% to more significant losses of greater than 50% over a 50mm length as seen in Figure 6.30. Steel edges along these zones transitioned smoothly in comparison to the abrupt and brittle section losses observed on longitudinal rebars and shear ligatures. A similar example is shown in Figure 6.31, which shows a severe case of corrosion on strands from the base tendon in Beam 17/3. Point A highlights a strand with complete section loss due to corrosion. Adjacent to this point, a thickening of the steel section was evident (Point B).

Typical profiles of corrosion-affected strands from Beam 17/4 are shown in Figure 6.32. Section losses are not as severe as those seen in Beams 17/3 and 118, however significant corrosion had initiated. Note the build-up of crusts over corrosion sites in conjunction with the onset of section losses. As Figure 6.25 shows, the strand condition at approximately 500mm from Diaphragm (v) worsened and cross-sectional area losses approached 30%. As previously mentioned, evidence of such corrosion was not visible at the concrete.
6.2 Steel Condition

(a) Longitudinal profiles

(b) Cross-sectional profiles

Figure 6.28: Examples showing strand profiles from areas of significant corrosion

Figure 6.29: In situ example of “flattened” strand profile subject to severe corrosion
6.2 Steel Condition

Figure 6.30: Strand profile with cross-sectional area variations due to corrosion

Figure 6.31: Insitu example of variable section losses

surface via propagation of cracks or spalling. Figure 6.32 shows examples where significant corrosion had occurred adjacent to zones of completely uncorroded bar (as observed on the conventional reinforcement).

Whilst it has been previously stated that general corrosion better describes the appearance of corroded strands, evidence of pitting was observed. In some regions, elongated pits were also observed at isolated locations within general corrosion zones. The depth of these pits ranged from superficial to significant (25% reduction in cross-sectional area), but losses were not as significant as pitting observed on conventional reinforcement. Figure 6.33 shows some typical examples of pitting on strands retrieved from Beams 17/3 and 118. Edges were not as rough or irregular as those observed on the conventional reinforcement, however drastic section losses were evident. In some areas, the pit edges were friable, but most had a clean surface along the base of the pit. In all cases, pitting was situated within significant corrosion and not directly adjacent to uncorroded steel (as observed on shear ligatures in Figure 6.15). Pitting was observed along some strands of Beam 17/4, however these were shallow (maximum pit depths were less than 1mm).
6.2 Steel Condition

Figure 6.32: Strand examples of corrosion adjacent to non-corroded regions

Figure 6.33: Examples of pitting profiles on prestressing strands
6.2 Steel Condition

and appeared to be in early stages of growth. Additionally, these pits typically appeared on the underside of strands where the base of the pits had a black or bright metallic appearance. The latter phenomenon will be discussed in more detail in Section 6.4.

Where possible, whole strand groups were retrieved from significant corrosion zones in order to observe corrosion patterns of the strands in situ. Figure 6.34 shows a core section taken through Diaphragm (iv) of Beam 118. The strand group was from the lower tendon, and significant corrosion was evident on the upper strands.

![Figure 6.34: Corrosion profile of strand group from lower tendon of Diaphragm (iv), Beam 118](image)

It was noted that strands completely surrounded by grout at the base of the tendon showed no section loss, however some black rimming had appeared around the circumference of these strands (Point A). In contrast, significant corrosion had occurred on the underside and adjacent sides of strands #1-6 (Point B). Section losses were evident on these strands, with strand #3 showing the most substantial loss. Irregular and jaggered edges within the corrosion zones were observed. Dense corrosion products had accumulated between these strands, which may have previously been air voids similar to those shown at Point C. Note the cracking denoted by Point D. According to conventional theory, the build-up of corrosion products can exert stresses on the adjacent concrete [26]. This is unsustainable
6.2 Steel Condition

by the concrete, which cracks to relieve the pressure. It is possible that the accumulation of corrosion products observed in Figure 6.34 may have caused the cracking observed at Point D. Observe the presence of a black-orange liquid between strands at Point E, which was accompanied by corrosion. It is thought that this liquid may be Ferrous Chloride, which is discussed further in Section 6.4.1.1.

A strand group with more advanced corrosion was retrieved from the base tendon of Beam 17/3 adjacent to Diaphragm (iv) (Figure 6.35). Note that the wrapping encircling the strands is tape, not a steel duct casing.

As for Figure 6.34, Point A designates strands at the base of the tendon that were in relatively good condition with no visible section losses. Strands at Points B and C show the opposite is true, with severe section losses observed on most strands along the upper edge of the tendon. Note the deep, concave nature of corrosion profile on strands # 4-7. The profile on strand #7 also has the appearance of steps (inset, Figure 6.35). Corrosion and
subsequent section losses appeared to have occurred on the bases of most upper strands, which may be due to voids in the grout at this location. Note at Point C flat edges were observed along both upper and lower quadrants of strands #1 to 3, similar to Beam 17/3 (Figure 6.28).

Strand failures were observed in more detail with the removal of strand groups from the concrete (Figure 6.36). Ductile failures were predominantly observed on strands from Beam 17/4 (Point A, Figure 6.36a). Strands from Beams 17/3 and 118 mostly failed in a brittle manner. Figure 6.36b shows the typical failure patterns from Beam 118. Observe the near-ductile failure at Point A in contrast to the brittle fracture at Point B. The failure pattern of strands in Beam 17/3 is shown in Figure 6.36c. Point A denotes a strand that appeared to failed in a brittle manner, which is consistent with observations from the literature where corrosion causes a reduction in ductility. Another brittle failure is shown at Point B. Observe the hairline crack which appeared to stem from the fracture point, denoted by this same point. A similar case is shown in Figure 6.37, which shows the fracture surface of a strand from Beam 118 after a tensile test. A hairline crack similar to that noted in Figure 6.36c is evident at Point A. Where a brittle fracture had occurred, the failure was located within or adjacent to a pit or a cross-section of significantly reduced area.

### 6.2.6 Diaphragm Strand Condition

Whilst the predominant focus of the present investigation is on the condition of the longitudinal prestressing strands for each beam, the condition of the transverse prestressing steel will now be briefly reviewed. It was noted in Chapter 2 that the load capacity of the Sorell Causeway Bridge relied heavily on the load transferring capabilities of the diaphragms; as such the condition of the transverse prestressing steel becomes a critical component for this to take place.

Some discussion regarding the condition of the grout in the transverse tendons has taken place in Chapter 4; in summary, the majority of the tendons were found to be fully grouted. In these tendons, the condition of the strands were observed to be good, with little or no surface rust. Some instances of minor grout deficiencies were observed however
6.2 Steel Condition

(a) Beam 17/4

(b) Beam 118

(c) Beam 17/3

Figure 6.36: Failure of strands due to corrosion

Figure 6.37: Cross-sectional view of strand failure from Beam 118
6.2 Steel Condition

it appeared that this did not have a significant impact on the corrosion of embedded strands. An example is shown in Figure 6.38; the size of the void is shown in Figure 6.38a. Note that the strands were completely encased in grout and no signs of corrosion was evident. It is thought that the cracks in the grout may be due to coring activities. Figure 6.38b shows the extent of the void, which is minuscule on this scale. However, also note the air bubbles imprinted along the top of the grout.

Some instances of severe grout losses were also noted, which was accompanied by corrosion. Diaphragm (iv) from Beam 17/3 exhibited the largest grout voids of all beams tested, and corrosion of the strands had commenced (see Figure 6.39). The extent of the void is shown in Figure 6.39a. Figure 6.39b shows the condition of the strands as the concrete was being broken open to reveal the steel. Whilst corrosion had obviously initiated across the strands, cross-sectional area losses were not severe with losses less than 20% of the strand area. Similar corrosion patterns were observed on these transverse strands to those of the longitudinal strands noted in the previous section.
6.3 Observation of Corrosion Products

6.3.1 General

The observation of corrosion products can provide useful information about the corrosion process itself and the participation of the surrounding environment. Due to the observation of a wide variety of corrosion products encountered during the course of the current work and their sometimes unusual nature, it was considered prudent to further investigate such products and their occurrence for each test beam. Section 6.3.2 provides some discussion on the formation of corrosion products within reinforced concrete and their subsequent decay over time. Section 6.3.3 details the methodology employed for this inspection process, and Sections 6.3.4 discusses the corrosion observations made.

6.3.2 Formation of Corrosion Products in Reinforced Concrete

6.3.2.1 Formation and Preferences

Reprising Chapter 4, the lowest energy state for iron at atmospheric conditions is as an oxide. Therefore the iron contained within steel has a natural tendency to revert to this stable state, which is facilitated by the presence of oxygen [87, 257]. When steel is em-
bedded in concrete, it will also attempt to return to this oxidised state, however the high alkalinity after initial cement hydration will facilitate the formation of a thin passive oxide layer (typically $Fe_3O_4$, $Fe_2O_3$ or both) which is self-renewing and protects the steel from degradation [57, 227, 286].

Modern theory suggests that the passive layer will completely break down in the event of carbonation or be destroyed locally by the build-up of chloride ions adjacent to the layer. The corrosion process can then proceed. As the corrosion process advances, corrosion products can form which depend on the availability of oxygen and water and the local pH. These stages are broadly represented by Figure 6.40, which is based on the literature documented by Marcotte [182] and Raina [232]. More detailed explanations on these processes and subsequent products are given in the following paragraphs.

The anodic and cathodic reactions for the corrosion process of steel in concrete is denoted by Equations 6.1 and 6.2 respectively, which has been reproduced from Chapter 3.

$$Fe \rightarrow Fe^{2+} + 2e^- \quad (6.1)$$

$$2e^- + 2H_2O + \frac{1}{2}O_2 \rightarrow 2OH^- \quad (6.2)$$

Hydroxyl ions ($OH^-$) will then subsequently react with the resulting ferrous ions ($Fe^{2+}$) to produce the first corrosion product, ferrous hydroxide ($Fe(OH)_2$) (Equation 6.3) [65, 159, 232, 274]. This product first forms in solution until its solubility is reached, resulting in a white or pale green precipitate. It exists under anaerobic conditions and where the pH is greater than 7. It is rarely observed, as it is extremely unstable and will oxidise quickly when exposed to atmospheric conditions [307].

$$Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2 \quad (white \ precipitate) \quad (6.3)$$

Where de-oxygenated conditions prevail, the Ferrous Hydroxide may transform into Ferric Oxide or Wustite ($FeO$) (Equation 6.4).

$$Fe(OH)_2 \rightarrow H_2O + FeO \quad (black) \quad (6.4)$$
6.3 Observation of Corrosion Products

Water is created as a by-product of this reaction. Wustite is black in colour and is a commonly found where localised corrosion has occurred in chloride environments [232]. However, Wustite is unstable at temperatures below 570° C [274], and thus Magnetite will preferentially form with the introduction of limited amounts of oxygen, which is also black (Equation 6.5). Again, these products can form in a moderately alkaline solution [102].
6.3 Observation of Corrosion Products

\[ 4FeO \rightarrow Fe + Fe_3O_4 \ (black) \]  (6.5)

Where oxygen and water are plentiful, the Ferrous Hydroxide will follow a different oxidation path to that described above. With the presentation of further oxygen, ferrous hydroxide will rapidly oxidise to an intermediate product, commonly known as Green Ruts [92, 102, 239]. These products were first identified in 1948 by G. Keller and remain the subject of many investigations [56, 92, 130, 202, 236]. Green rusts are commonly associated with marine environments. As the name suggests, these products are green in colour and comprise complex hydroxyl-salts with varying proportions of ferric and ferrous ions \((Fe^{2+}\ and\ Fe^{3+} \) respectively). They have the following structure (Equation 6.6):

\[ [Fe_{x}^{III}Fe_{y}^{II}(OH)_{3x+2y-2}][A^{-}]_{z} \ (green) \]  (6.6)

where \(A^{-}\) is a salt that is incorporated into the compound, and is designated as either GR(I) or GR(II). GR(I) salts include Flouride \((F^{-})\), Chloride \((Cl^{-})\), Bromide \((Br^{-})\) or Iodide \((I^{-})\) ions; GR(II) currently represents lesser amounts of sulphate \((SO_4^{-})\) and carbonate \((CO_3^{-})\) ions [92]. In reinforced concrete media, chloride and carbonate ions are the preferential salts [128].

Green Ruts are notoriously unstable and will quickly oxidise when introduced to atmospheric oxygen [238]. They will subsequently form Ferric Hydroxide, \((Fe(OH)_3)\), shown in Equation 6.7.

\[ Fe(OH)_2 + O_2 + 2H_2O \rightarrow Fe(OH)_3 \ (brown) \]  (6.7)

This product is unstable and will subsequently lose water to form either a hydrated form of Ferric Oxide \((FeOOH, \ commonly\ known\ as\ iron\ oxy-hydroxides)\) or Maghemite, a form of Magnetite\((Fe_2O_3, \ or\ commonly\ known\ as\ “red\ rust”)\). The presence of both Magnetite and Maghemite, as well as Haematite have also been recorded in the literature for aged corrosion products [102, 159, 274]. The formation of Ferric Oxide and Maghemite is represented by Equation 6.8 and 6.9 respectively.

\[ Fe(OH)_3 \rightarrow H_2O + FeOOH \ (colour\ varies) \]  (6.8)

\[ 2Fe(OH)_3 \rightarrow 3H_2O + Fe_2O_3 \ (red) \]  (6.9)
6.3 Observation of Corrosion Products

There are various forms in which the hydrated Ferric Oxide can take in reinforced concrete, all of which result from the intermediate Green Rust phases [92, 102, 182, 186, 202, 237]. These include:

- Goethite \( (\alpha - FeOOH) \)
- Lepidocrocite \( (\gamma - FeOOH) \)
- Akaganeite \( (\beta - FeOOH) \)

Goethite and Lepidocrocite are amongst the most common corrosion products found in deteriorating concrete structures in varying quantities [94, 111, 219, 269]. Magnetite and/or Maghemite are also present, which have similar structures according to Cornell & Schwertmann [101]. In certain circumstances, Lepidocrocite is found in more abundance than Goethite, especially where lower levels of chlorides exist as it will form preferentially over Goethite [188, 237]. However Lepidocrocite will decay to Goethite over time where temperature conditions and an abundance of ferrous ions \( (Fe^{2+}) \) exist, as Goethite is one of the most stable oxides, which may in turn then transform into Magnetite [101, 162]. Akaganeite is typically present where high chloride concentrations exist, such as marine environments [212]. Akaganeite can form under slightly acidic conditions, where the pH is between 4 and 6 [102, 237].

Table 6.3 shows a summary of findings by Genin et al [127] and Refait and Genin [236] which show the likely formation of corrosion products in relation to the environment [182]. Genin and colleagues state that the formation of rust products will ultimately depend on the \( [Cl^-]:[OH^-] \) ratio existing within the system; increasing concentrations of chlorides will lead to preferential formation of Akaganeite and intermediary Green Rusts, in lieu of stable end products such as goethite and magnetite.

6.3.2.2 Corrosion Product Migration and Layers

Where corrosion occurs on rebars in reinforced concrete, corrosion products can build up against the steel and also migrate away from its origin through the concrete. The literature is sparse when discussing how far corrosion products can travel from its point of origin, however it appears that it is a function of the porosity of the concrete, the corrosion product type and the tensile strength of the concrete [11, 12, 182]. Corrosion
6.3 Observation of Corrosion Products

Table 6.3: Preferential formation of corrosion products with varying $[Cl^-]:[OH^-]$ ratio [127, 182, 236]

<table>
<thead>
<tr>
<th>[Cl$^-$]:[OH$^-$] Ratio</th>
<th>Initial Product</th>
<th>Intermediate Product</th>
<th>End Product(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 1$</td>
<td>$Fe(OH)_2$</td>
<td>-</td>
<td>Goethite, Magnetite</td>
</tr>
<tr>
<td>1 - 1.025</td>
<td>$Fe(OH)_2$</td>
<td>-</td>
<td>Magnetite (hydrated)</td>
</tr>
<tr>
<td>1.025 - 1.11</td>
<td>$Fe(OH)_2$</td>
<td>Some GR(I) (Cl$^-$)</td>
<td>Magnetite</td>
</tr>
<tr>
<td>1.11 - 1.75</td>
<td>$2Fe(OH)_2$</td>
<td>GR(I) (Cl$^-$)</td>
<td>Lepidocrocite</td>
</tr>
<tr>
<td>$\geq 1.75$</td>
<td>$2Fe(OH)_2 \cdot FeOHCl$</td>
<td>GR(I) (Cl$^-$)</td>
<td>Lepidocrocite, Goethite, Akaganeite</td>
</tr>
</tbody>
</table>

Products will accumulate in voids adjacent to activated anodic sites on the steel, as shown in Figure 6.41. The voids are likely to be air voids or voids left by dissolved calcium hydroxide.

![Figure 6.41](image)

(a) Initiation, prior to cracking  
(b) Build up of corrosion products, causing cracking

Figure 6.41: Accumulation of Corrosion Products in voids adjacent to anodic sites [11, 12]

The amount of space the corrosion products occupy will depend on its composition, as the products vary in molar volume. This relationship is shown in Figure 6.42; it shows that corrosion products in an oxygen-deprived scenario (such as Wustite or Magnetite) have volumes similar to that of base iron. Alternatively, products that result from an abundant supply of oxygen and water will occupy a volume up to six times this amount.
The increasing volume has implications on the surrounding concrete. Whilst concrete performs well in compression, it is relatively weak in tension. With the volumetric increase afforded by corrosion products, radial stresses are exerted on the surrounding concrete. These stresses are unsustainable by the concrete and subsequently microcracks form adjacent to anodic zones. Corrosion products can then infiltrate these new areas, and the corrosion process continues. The build-up of corrosion products will eventually lead to cracking through the entire depth of cover and spalling will occur (Figure 6.41b). New and existing areas of steel are thus exposed to atmospheric conditions and the corrosion process can continue unrestricted [21, 67, 74, 188, 221, 269, 290]. It is noted that Figure 6.42 should only serve as a volumetric comparison between the corrosion products.

A number of authors have made attempts to determine the typical corrosion products within this zone, often referred to as the steel-concrete interfacial zone [94, 111, 137, 226, 310]. Corrosion layers in concrete can be divided into three zones: Steel, Intermediate and Concrete. The intermediate zone is separated into a further two zones: the inner layer (in

Figure 6.42: Estimated relative volumes of corrosion products with respect to metallic iron [188, 247]
direct contact with the steel) and the outer layer (in direct contact with the concrete). In summarising results on the intermediate zone from the aforementioned references, dense Magnetite/Maghemite comprises the internal corrosion layers which then transform to more porous iron oxyhydroxide varieties (FeOOH) towards the outer layers. Calcium, silicon and quartz may also be found in the outer layer due to its proximity with the concrete.

6.3.3 Methodology

Several representative steel samples were retrieved from various locations in each beam in order to identify typical corrosion products. Standards reviewed for sample collection and analysis were ASTM G46-94 and the NACE RP-01-73 [45, 207]. A variety of methods were employed to analyse these samples; initially, all samples were subjected to a visual inspection which can often provide useful information (for example, the study by Virmani et al [307]). This included observing samples under a high-powered microscope and obtaining detailed photographic images.

Samples were then subjected to more detailed analysis for the characterisation of corrosion products. The most commonly used analysis techniques for steel and reinforced/prestressed concrete applications are X-Ray Diffraction (XRD), Scanning Electron Microscopy (SEM) and Energy Dispersive X-Ray Spectroscopy (EDS). These techniques have been reviewed in detail by Bungey and Millard [77] and Cornell and Schwertmann [104]. For the current project, both bulk and in-situ samples were analysed. XRD was predominantly utilised for analysing bulk samples, whereas SEM was mainly used for in-situ samples. The following sections will now review some of the key observations made in relation to corrosion products found from the three test beams.

6.3.4 General Observations

A variety of corrosion products were observed. Figures 6.43 to 6.47 show representative examples of corrosion products observed in- and ex situ. Detailed inspection showed that the majority of the rust staining observed on the concrete and at the surface was attributable to corrosion of the reinforcement rather than the prestressing strands. Corrosion products from the former ranged in colour and oxidation state, ranging between brown hues, black, deep red, various shades of green, and dull to bright orange and yellow.
6.3 Observation of Corrosion Products

Products observed on the strands were predominantly black and green on initial exposure. All corrosion products faded to brown and red over time.

Figure 6.43: Accumulation of corrosion products adjacent to corroding rebar

Figure 6.43 shows a representative sample of a corroding longitudinal rebar. Corrosion products had initially accumulated against the bar, but had subsequently migrated away through the concrete. Note the chalk-white powder bordering the products; this phenomenon was observed most frequently adjacent to corrosion zones from Beam 118. This powder was not analysed for composition but it is likely to be a form of calcium carbonate. An example of typical corrosion products observed on prestressing strands is shown in Figure 6.44. The different stages of oxidation are visible, denoted by orange and black zones in the macroscopic image in Figure 6.44b.

Figure 6.44: Corrosion product built up over strand

Also commonly observed were zones of crusting and corrosion on strands from the upper region of the tendons (Figure 6.45). This occurred on both the upper and lower surfaces
6.3 Observation of Corrosion Products

of the strands, especially on bars from significant corrosion zones. For the example shown in Figure 6.45, the corrosion products were accumulating on the underside of the strands in contact with the grout. In this instance, corrosion products appeared to build-up in existing grout voids at this location (see previous discussions relating to grout voids in Chapter 4). Note the morphology of the corrosion products intermixed with grout residue, which resembles the appearance of “burst bubbles”.

Figure 6.45: Corrosion product build-up on underside of strand group from Beam 17/3

Portions of these corrosion products were removed and these are shown in Figure 6.46. Note the density and volume of the products shown in Figure 6.46b; note also the incorporation of apparent air voids amongst the products. In Figure 6.46b, the corrosion products appeared to take on the shape of adjacent prestressing strands. The case shown in Figure 6.46c shows the accumulation of dense corrosion products which have formed a hollow section. In other cases, the accumulation of corrosion products had a layered appearance (Figure 6.47).

XRD analysis revealed varying results. For longitudinal reinforcement with little or no rust, the XRD analysis identified phases (in decreasing order) of Iron, Magnetite, Haematite, Lepidocrocite and Wustite. Minor phases of Calcite, Portlandite and Quartz were also identified, which most likely relate to cementitious materials. Where corrosion was significant, the XRD analysis revealed Magnetite as the predominant phase, as well as Goethite and Lepidocrocite. Akaganeite and Iron (III) Oxide Chloride ($FeOCl$) were identified as minor phases in some cases, although where corrosion was severe Akaganeite
6.3 Observation of Corrosion Products

(a) Example #1  (b) Example #2  (c) Example #3

Figure 6.46: Corrosion product from prestressing strands

Figure 6.47: Layers of corrosion product

was found to be more dominant. Similar observations were made on severely corroded prestressing strands. Predominant phases included Magnetite, Goethite and Lepidocrocite, with an increasing prevalence of Akaganeite in areas of poor condition.

Many of the phases identified by XRD analysis were aged oxides (i.e. corrosion products exposed to a continuous supply of oxygen and water). Therefore, attempts were made to identify corrosion products with minimal oxidation, which included green and black rusts. These products were difficult to officially identify due to their rapid oxidation. However, samples successfully analysed by XRD revealed predominant phases of Akaganeite, Green
Rust (I), and a variety of Iron Oxy- and Hydroxyl Chlorides (such as \(FeOCl\)). These phases are discussed in more detail in Sections 6.3.4.2 and 6.3.4.3.

### 6.3.4.1 Corrosion Product Migration

As noted previously, most corrosion products appeared to originate from the conventional reinforcement. Figure 6.48 shows an example of corrosion product travel, with the predominant point of origin being either shear ligatures or longitudinal rebars. The distances the corrosion products had migrated varied, with some products travelling maximums of 100mm from the point of origin. The presence of a similar white, chalk-like residue was again observed bordering corrosion products along the base of the beam. The migration of the corrosion products predominantly followed existing cracks or defects (such as longitudinal web cracking or soffit cracking adjacent to Diaphragm (iv) for Beams 17/3 and 118). Travel distances were considerably more significant in Beam 118 in contrast to Beams 17/4 and 17/3.

Consider, in comparison, the travel of corrosion products from the post-tensioning strands as shown in Figure 6.49. Figures 6.49a and 6.49b show typical examples of the extent to which corrosion had spread along the upper surface of the tendon. However, only a small percentage of this corrosion product is evident outside the confines of the tendon. This may be due to the leaching out of rust products in a soluble form (for example, dissolved \(Fe^{2+}\) and \(Cl^-\) as detailed in Chapter 3). This is discussed in more detail in Chapter 7.
6.3 Observation of Corrosion Products

Figure 6.49c shows the progression of corrosion products along the tendon interface; note the dense concrete matrix which shows the formation pattern of the tendon.

Figure 6.49: Examples of corrosion product migration (prestressing strands)

6.3.4.2 Green Rusts [GR(I)]

Green rusts were observed on both conventional reinforcement and post-tensioning strands from all beams. Incidence frequency increased in areas where corrosion was severe. The colour varied from bright coppery-green and blue-green hues to dull, dark greens which were closer in shade to black than green.
6.3 Observation of Corrosion Products

Figure 6.50 shows examples of green rusts observed on shear ligatures located in zones where cracking was most severe. The rusts were predominantly located in or rimming the perimeter of deeper pitting. A colour difference is observed between the blue-green pits shown in Figure 6.50a and the copper-green colour shown Figure 6.50b. Black rusts were noted to occur simultaneously with the green rusts on a number of occasions. Evidence of a darker green/black rust within an elongated pit on diaphragm reinforcement is shown in Figure 6.51. Older oxidation products were observed to the right of the pit. Figure 6.52 show similar examples of similar bright green rusts in pits along strands.

(a) Blue-green colour
(b) Copper green colour

Figure 6.50: Examples of Green Rusts observed on shear ligatures

Figure 6.51: Dark green rust occurring on pitting on diaphragm shear ligature

Figure 6.52: Examples of Green Rusts observed on prestressing strands
6.3 Observation of Corrosion Products

Instances of green rusts were also recorded within corrosion products staining adjacent concrete. For example, Figure 6.53 shows green rust accumulating amongst corrosion products that have emanated away from a shear ligature in Beam 17/4. Similar observations were made for corrosion products adhering to upper regions of tendons. XRD analysis of representative green rust samples confirmed the presence of the chloride-based Green Rust (I). Other phases identified were the chloride complex Iron Chloride Hydroxide ($\text{Fe}_6\text{Cl}_{2-\delta}(\text{OH})_{12+\delta}$), Hibbingite ($\alpha - \text{Fe}_2(\text{OH})_3\text{Cl}$), Iron (III) Oxide Chloride ($\text{FeOCl}$) and Akaganeite.

An unusual green rust was observed on a section of longitudinal reinforcement from Beam 17/3. As shown by Figure 6.54, an opaque, shimmery green layer overlay the surface of a relatively smooth steel surface that had obviously corroded. It was associated with thin layers of black and orange-brown rusts. It had a wet appearance and there were no other voluminous corrosion products in this vicinity. Severe section losses were observed along the base of the rebar and the adjoining shear ligature adjacent to this location. The significance of this observation is not known, as there was no opportunity to conduct a more detailed analysis due to rapid oxidation. As such, it cannot be said conclusively that this observation is associated with GR(I).

When retrieving severely corroded steel sections, pale green and white substances were occasionally observed amongst corrosion products on prestressing strand samples. Examples of these are shown in Figure 6.55 and are not the same products as those observed on Figure 6.43. These “white” products oxidised extremely quickly to deeper green and
black products. They were not visible after 5 minutes of exposure to atmospheric con-
ditions. Due to this rapid oxidation cycle, samples could not be obtained for XRD analysis. However, it may be postulated that this product was representative of the unstable white precipitate Ferrous Hydroxide \((Fe(OH)_2)\), the foundation of all iron oxides, as discussed previously.

6.3.4.3 Black Rusts

The observation of black rusts observed appeared to fall into two broad categories: dry, black rusts and moist/wet black rusts. For both scenarios, the black rust oxidized rapidly to form other oxidation products that were initially bright orange and transformed to dark brown products. The following paragraphs detail the findings for each category.
6.3 Observation of Corrosion Products

**Dry, Black Rust**

Dry, black rusts were observed on various steel surfaces, with only limited observation of them on and through adjacent concrete. Typical examples found across prestressing strands are shown in Figure 6.56. Black rusts were observed to be predominantly confined to elongated pits across the steel surface, however this rust was also observed extending over surfaces affected by thick corrosion products. It was often observed surrounding areas of green rusts, as discussed in Section 6.3.4.2.

![Figure 6.56: Examples of dry, black rust](image)

An example of dry, black rust observed across a pitted surface on conventional reinforcement is shown in Figure 6.57a. The surface had a slight “shimmery” appearance. This example has been discussed previously in relation to the irregular pitting pattern (Figure 6.13), however it is also representative of dry, black rust. Figure 6.57a shows the steel immediately after it was retrieved from the concrete. 24 hours later, this pit had oxidized to a dark brown colour (Figure 6.57b). An XRD analysis of the initial oxidation state revealed a prevalence of Magnetite with traces of Maghemite and Iron (III) Oxide Chloride.

![Figure 6.57: Dry, black rust adhering to a pit on shear ligature from Beam 17/3](image)
A series of black “spots” appeared on several strands from Beam 17/3 upon exposure to aerobic conditions (Figure 6.58). These appeared to form in small indentations across the strand surface, presenting a black, powdery appearance. These spots disappeared within hours of exposure.

![Figure 6.58: Small, powdery, black spots covering the surface of steel recently exposed to the atmosphere](image)

**Wet, Black Rust**

It appears that only some authors in the literature have made reference to or observed wet, black rust [65, 89, 204]. Slimy, black deposits are also associated with Sulphate Reducing Bacteria [59, 103, 172, 306]. For the present investigation, several instances of wet, black rust were observed in all beams, primarily towards the base of the beam and within the confines of tendons. It was observed in both isolated instances or combined with other corrosion products, with more frequent sightings of this rust in zones of pitting. As the name suggests, it has an intensely black and wet appearance. It was not voluminous, but comprised a very thin layer, non-discernible to the naked eye.

Figure 6.59 shows examples of wet, black rust observed. Figure 6.59a shows a typical example of pitting that had occurred on a shear ligature, which was covered with wet, black rust. There was little or no corrosion observed either side of this pit. Figure 6.59b shows an example where wet, black rust was observed in a pit on the surface of a strand, which extended into adjacent voids furnished by the grout. Figure 6.59c shows an isolated pit on a strand that is filled with a translucent, black fluid. The presence of small bubbles was observed in this fluid, which had a slightly slippery feel when wiped away.
Upon removal of the fluid, a smooth, rust-free, clean pitted surface was observed beneath. This was not an isolated incident; several other sites exhibiting wet, black rust were observed to either mask or partially cover a clean, bright metallic surface. The first example is shown in Figure 6.60. Several spots of wet, black rust were observed, and on closer inspection these spots covered clean metallic surfaces.
6.3 Observation of Corrosion Products

Consider Figure 6.61a; a similar isolated instance of wet, black rust was observed covering a bright metallic, clean surface marked by small, concentric pits. This section of longitudinal reinforcement was uncovered from Beam 118, and within 24 hours had oxidised to the bright orange products observed in Figure 6.61b, masking the metallic surface. The majority of these cases were accompanied by a slightly metallic odour upon retrieval from the concrete. Such instances of pitting were noted to occur more frequently on conventional reinforcement, although reference is made similar pitting morphologies noted on prestressing strands.

![Figure 6.61: Wet, black rust covering a bright, metallic pitted surface](image)

A final observation is shown in Figure 6.62. A pocket of black, “oily” rust was observed located in concrete voids adjacent to a corroding shear ligature retrieved from Beam 17/4. No odour was detected, however the liquid had a slightly “frothy” appearance. This instance was not isolated. An XRD analysis of such samples was not physically possible. However a series of microbiological tests were conducted on these sites, which included indicator tests for Sulphate Reducing Bacteria (SRB) and Iron Related Bacteria (IRB). Tests for SRB were inconclusive but all sites tested positive for the presence of IRB. These tests are discussed in more detail in Section 6.4.3.
6.4 Other Observations

6.4.1 Aggregate rims/corrosion accumulation

During experimental work, some anomalies were observed regarding the basalt aggregate from some samples. These are now described. Discolouration was observed on isolated aggregate pieces adjacent to steel corrosion sites, where it appeared that corrosion products had accumulated around the perimeter of the aggregate. Figure 6.63 show some representative examples. Various iron oxide colours were observed to surround and cover the aggregate surface (Figure 6.63a and 6.63c); a black boundary layer was also visible (Figure 6.63b and 6.63c). Fine cracks was observed in some locations at the aggregate/cement paste interface. To the author’s knowledge, there is only one other publication that has observed a similar phenomenon. Aligizaki et al [10] observed the accumulation of iron oxides around the circumference of limestone aggregates as part of a larger study based on cracking due to corroding reinforcement. The significance of this finding is not known in this context.
6.4 Other Observations

(a) Example #1

(b) Example #2

(c) Example #3

Figure 6.63: Corrosion products accumulating around the perimeter of aggregate

Another example is shown in Figure 6.64. Whilst retrieving a core from Beam 118, two pieces of aggregate towards the base of the core were noted to be covered in a thin, orange/brown layer which had the same appearance as iron oxide. A greater proportion of this layer was noted rimming the aggregate pieces. A brown, glassy liquid was observed forming between the two pieces. The cause for these observations is unknown. It is unlikely that the rimming had occurred due to the corrosion of adjacent reinforcement, as there was no reinforcement present at this location. It is speculated that a small section of tie-wire may be at fault, perhaps located between the aggregate pieces. The brown liquid had a similar appearance to Ferrous Chloride ($FeCl_2$), which is discussed in Section 6.4.1.1.
White deposits were observed along the aggregate edges in isolated locations. Figure 6.65 shows some examples. Whilst these observations appeared to be randomly and sparsely distributed throughout the concrete, they were observed more frequently in sliced core samples after they had been submerged and boiled for 12 hours (Chapter 5). This suggests that the aggregate may be slightly reactive. Tests conducted by Koranc and Tugrul [165] suggest that basalt can exhibit signs of alkali-aggregate reactivity (AAR), however olivine basalt is more likely to show very low levels of reactivity. An XRD analysis of the white deposits identified a predominance of calcite, with phases of vaterite and aragonite; no AAR gel was detected. It may be postulated that the calcite layers were formed in void spaces or microcracks at the aggregate/cement paste interface. Carbonation may also have been influential in its formation (see Chapter 5).
6.4 Other Observations

6.4.1.1 Iron Chlorides

During the course of corrosion investigations, small, spherical beads of clear, orange-yellow liquid were occasionally observed between steel/concrete interfaces of freshly cut concrete surfaces. Initially, the beads had a glassy appearance, but would form a crystalline, orange-brown glaze over the top of the liquid over time. In some cases, the beads had dried out completely to form a fragile, orange-brown crystalline crust.

This phenomenon is well known and documented by archaeologists, where its formation is often referred to as “chloride weeping” [178, 256, 284, 311], “beading” [84, 106] or “sweat-ing” [130, 183, 213]. The liquid itself comprises aqueous iron (II) chloride compounds (ferrous chloride or FeCl$_2$) and minimal quantities of Fe$^{3+}$ in solution. It is acidic in nature, with pH ranging between 1 and 4. Whilst the exact mechanisms of the weeping phenomenon is unknown, current theory suggests that it results from the hygroscopic nature of the iron chloride salts, and initiates when the steel dries out rapidly or there are fluctuations in relative humidity [106, 183, 255]. The skin that forms over the ferrous chloride liquid is thought to be Akaganeite [213].

In archaeological practice, usually only small amounts of ferrous chloride are observed as it dissociates readily in the presence of water, and subsequently oxidises to a lower energy state [106]. For engineering investigations, it is postulated that similar reasons may account for the lack of direct observation of such compounds for reinforced concrete.
structures. The iron chlorides may also have been incorporated into other corrosion products (such as Akaganeite or Lepidocrocite) or, as a result of the high solubility of the ferrous chlorides, have simply been washed away [223].

Some examples of this phenomenon are now described. Most cases of iron chlorides were noted on Beams 17/3 and 118, with little evidence found on sections from Beam 17/4. Frequency and quantities increased in zones of significant corrosion and cracking, with sections from Beam 118 being the worst. The evolution of “chloride weeping” is evident in Figure 6.66. Here, iridescent, yellow liquid weeping was observed from the steel/concrete interfaces approximately 24 hours after the concrete surface was cut. The liquid was translucent, and dark yellow to orange in colour. Some oxidation of the liquid had occurred, leaving a crystalline crust staining the surface of the concrete. In Figure 6.66a, the ferrous chloride liquid appears to have emanated from a crack in the mass concrete, and not from the interface between the reinforcing bar and the concrete.

(a) Example #1

(b) Example #2

Figure 6.66: Examples of Ferrous Chloride weeping from steel/concrete interfaces
6.4 Other Observations

Figure 6.67 shows close-up views of weeping at various stages in time. Firstly just after formation (Figure 6.67a) and secondly older beads which have completely dried out (Figure 6.67b). Note the collapsed shell in Figure 6.67b - these were found to be very fragile. In situ examples of the formation process are shown in Figure 6.68. Similarities were observed between the corrosion products that had formed along the top of the prestressing strand (Figure 6.68b) and those noted from general corrosion observations made in the present investigation (Section 6.2). There was also clear evidence of beading on the steel surface itself (Figure 6.67a).

Figure 6.67: Images showing the aging process of ferrous chloride beads

Figure 6.68: In situ examples showing the formation of ferrous chloride
A severe case is shown in Figure 6.69. Significant amounts of ferrous chloride were produced from concrete and/or steel interfaces from the base tendon of Beam 118 over Diaphragm (iv). The weeping was observed at different stages of maturity; there was the early evolution of yellow beads, an intermediate stage where the liquid was contained within a thin, dark brown/orange crystalline shell, and the final stage where dehydrated crystalline crusting was observed adjacent to the surface of the steel.

Figure 6.69: Severe case of “chloride weeping” from prestressing strands in Beam 118

A sample of the liquid was collected for XRD analysis and observation by SEM. The liquid sample shown in Figure 6.70a was a cloudy, green colour. Older oxidation products of the liquid were also evident in the form of crystalline flakes of black and bright orange. The XRD analysis identified Iron Chloride Hydrate ($FeCl_2 \cdot 4H_2O$) as the predominant phase, along with an iron chloride hydroxide complex, $Fe_6Cl_{2-x}(OH)_{12-x}$. Other XRD analyses carried out at various times throughout the project had also detected the presence of Hydromolysite, a hydrated ferric chloride ($FeCl_3 \cdot 6H_2O$).

Dried crystalline products were retrieved from this liquid sample (Figure 6.70a) and for analysis via SEM. The SEM image is shown in Figure 6.70b. A predominance of cubic crystals was observed. Two distinct phases were evident, highlighted in Figure 6.70b by Points A and B. A composition analysis using Energy Dispersive Spectrometry (EDS) was conducted at both points, which are shown in Figure 6.71. The lighter crystal structures
denoted by Point A showed very high levels of chlorides and some iron to the exclusion of other elements (see Figure 6.71a), which was consistent with the Iron Chloride Hydrate detected by the XRD analysis. The analysis for Point B produced a similar pattern with a greater proportion of iron (see Figure 6.71b). A distinct sulphur peak was also observed in this pattern.

Figure 6.70: SEM image for representative ferrous chloride sample

Figure 6.71: EDS analysis of Points A & B from Figure 6.70b
6.4 Other Observations

6.4.2 Bright, Metallic Pitting Surfaces

As briefly mentioned in Sections 6.2.2 and 6.3.4.3, instances of bright, metallic surfaces were identified on steel. This section provides further examples pertaining to this topic. Different morphologies of this pitting were observed on the prestressing strands and conventional reinforcement; these ranged from isolated and elongated pits, concentric and stepped pits, or bright, gold/bronze-coloured surfaces containing smaller pits. These morphologies are now discussed.

6.4.2.1 Prestressing Strands

Discussion pertaining to the observation of large and elongated pits along the top surfaces of the prestressing strands can be found in Section 6.2.3. Similar pits were also observed on the underside of these strands at some locations, in direct contact with the tendon grout. These pits were mostly elongated and localised, with the base of the pit showing a bright, metallic surface. No corrosion product was observed overlaying this surface. The majority of strands retrieved from the upper regions of the tendons exhibited varying degrees of this pitting morphology. Beam 17/4 had the greatest number of sightings, however it is possible that similar instances may have been found on Beams 17/3 and 118 but may have been obscured due to the larger quantities of corrosion products.

Typical examples are shown in Figure 6.72. The pits varied in colour, ranging from bright, silver bases rimmed by black rust (Figure 6.72a), a gun-metal grey, shimmery surface also rimmed by black rust (Figure 6.72b), or a bronze or copper coloured surface with an assortment of corrosion products accumulating at the pit edge (Figure 6.72c). Observations of the latter case were less common and would sometimes occur even when there was no other evidence of adjacent corrosion. The morphology of these pits were mostly elongated and had a smooth bright metal surface at the base of the pit. “Steps” or rings were also observed in some pits. Pit depths varied from very shallow (< 0.1mm) to significant (approximately 1-2mm).
6.4 Other Observations

(a) Bright, metallic surface

(b) Gun-metal grey, shimmery surface

(c) Bronze/copper-coloured surface

Figure 6.72: Bright metallic surfaces at the base of pitting on prestressing strands
XRD analyses on representative bright, metallic pit surfaces revealed a high prevalence of pure Iron and Magnetite, with lesser phases of Akaganeite and Goethite. Minor phases of Gypsum and Quartz were also identified, presumably from the adjacent grout. Observation of these samples by SEM with corresponding EDS scans are shown in Figure 6.73. The back-scattered electron (BSE) images for Points A and B revealed elemental differences between the bright, metallic surfaces and the darker corrosion products, which are most likely to be the differences between the pure iron (bright surfaces) and the subsequent iron oxides (darker surfaces). The EDS analyses verified the prevalence of iron (as shown in Figure 6.73).

A close-up view of Point B is shown in Figure 6.74. The appearance of fine “flow lines” was evident on the raw metal surface. An intersecting pair of cracks or notches were also observed at the centre of the pit, in which various iron oxides had accumulated. It was observed that the oxidation rate of some of these pits was very slow; the bright, metallic surface was still recognisable after 1 month of exposure to atmospheric conditions.

6.4.2.2 Conventional Reinforcement

Similar pitting observations were observed on some regions of conventional reinforcement. The pitting morphology was slightly different to that observed on the prestressing strands, with pits observed to be either elongated and “flat” or deep with smaller pits at the base, both with bright metallic bases. A ringed structure was an additional common observation.

An example of the elongated, flat pitting morphology is shown in Figure 6.75; The surface of the bar at these locations was either rust-free or obscured by normal corrosion products. The steel surface itself was smooth and had a silvery-gold-coloured appearance. It was also generally free of other corrosion products with the exception of the example shown in Figure 6.75b, which is partially covered in a wet, black rust.

The extent of these bright surfaces varied and were predominantly observed on shear ligatures retrieved from crack or spall zones in the web. No significant section losses were observed directly in these areas, however in some adjacent regions severe section losses
Figure 6.73: Observation by SEM of bright, metallic pit bases on prestressing strands

Figure 6.74: Close-up of bright, metallic surface on Point B from Figure 6.73
6.4 Other Observations

(a) Example #1

(b) Example #2

Figure 6.75: Examples of flat, metallic surfaces on shear ligatures

were recorded (as is observed on Figure 6.75a).

As previously mentioned, deep pits with a bright metallic bases were also observed, with silvery-gold coloured surfaces similar to that observed in Figure 6.75. The morphology of the pit itself was either a series of micro-pits having formed within the main pit or as a series of concentric steps or rings. When bars containing such pits were retrieved from the concrete, the pitted surface was either covered by a wet, black film (which had a slight metallic odour) or was completely un-obscured by corrosion products (i.e. the bright, metallic surface of the pit was immediately obvious). Figure 6.76 shows an example previously discussed in Section 6.3.4.3 for the observation of wet, black rusts. Note at Point A that a bright, metallic surface was found under the black rust. The surface itself was marked by a series of smaller, concentric pits less than 0.5mm in diameter. Several instances of this pitting morphology were observed on several longitudinal rebars and shear ligatures. An XRD analysis of the wet, black rust overlaying a shiny, pitted surface on a separate shear ligature is shown in Table 6.4. It identifies akaganeite as the predominant phase; note that minor traces of iron sulphide were also detected.
6.4 Other Observations

Figure 6.76: An example of bright, metallic, pitted steel surface obscured by wet, black rust

![Image of pitted steel surface with wet, black rust covering it.]

Table 6.4: XRD analysis of wet, black rust overlaying a smooth, silver pitted area

<table>
<thead>
<tr>
<th>Phase Priority</th>
<th>Compound Identified</th>
<th>Chemical Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main</td>
<td>Akaganeite</td>
<td>$\beta - FeOOH$</td>
</tr>
<tr>
<td>Semi-dominant</td>
<td>Wustite</td>
<td>$FeO$</td>
</tr>
<tr>
<td>Iron</td>
<td></td>
<td>$Fe$</td>
</tr>
<tr>
<td>Minor</td>
<td>Magnetite</td>
<td>$Fe_3O_4$</td>
</tr>
<tr>
<td>Lepidocrocite</td>
<td></td>
<td>$\gamma - FeOOH$</td>
</tr>
<tr>
<td>Possible Minor Phase</td>
<td>Iron Sulphide</td>
<td>$FeS$</td>
</tr>
</tbody>
</table>

Figure 6.77 shows an example of a smooth and shiny pitted surface that was retrieved “as-is” from the concrete, with no corrosion product covering the pit surface. It is representative of a small number of examples retrieved in this condition. The surface was gold in colour, and showed a number of smaller, concentric pits at apparently random locations 6.77a. Figure 6.77b is an SEM image of the same region, showing the smooth, irregular surface which appeared to be integrated with, or undercut by, adjacent iron oxide surfaces. Some of the iron oxide edges adjacent to the pit were rough and irregular, as if broken. This latter observation perhaps suggests that the pit was once covered in corrosion product and inadvertently removed during the demolition process. An EDS analysis of the overall pitted surface indicated a predominance of pure iron.

Figure 6.78a shows a close-up SEM image of the pit surface at Point A from Figure 6.77b. It had a grainy texture but was consistently smooth; little else was observed on the surface with exception of one small “nodule” at the centre of the scan (Point A). An EDS scan of
6.4 Other Observations

(a) Overview of pit insitu

(b) SEM image of pit

Figure 6.77: Examples of flat, metallic surfaces on shear ligatures

the nodule (and adjacent silver surface) revealed a high prevalence of iron with traces of potassium, chlorine, silicon and sulphur (Figure 6.78b). The same smooth, grainy surface was observed at an adjacent spot (Figure 6.79a), with a shard-like crystal and a tube-like structure overlaying the surface (Points A and B respectively). Iron, Sulphur, Chlorine and Calcium were the predominant elements identified by EDS analysis (Figure 6.79b).

Figure 6.80a shows an SEM surface image of the outer edge of the pit in Figure 6.77, showing a compound with the appearance of a fibrous formation. This formation appeared to be intertwined amongst other corrosion products. An EDS analysis identified high levels of Chlorine, followed by Sulphur, Calcium, Iron and Aluminium (Figure 6.80b). Minor levels of Chromium were also detected. An XRD analysis of the entire pitted surface from
Figure 6.77 showed a predominance of iron, with minor phases of Magnetite and Goethite (Table 6.5). A hydrated iron sulphate hydroxide, Xitieshanite, was identified as a possible minor phase, which can range in colour from a bright yellow-green or a brownish-red if oxidised [289].
6.4 Other Observations

Isolated pits with a stepped morphology were identified on several conventional reinforcement bars. A good example is shown in Figure 6.81. This circular, stepped pit was located on a shear ligature from Beam 118 adjacent to the longitudinal web crack on Face D. There were no adjacent section losses or accumulation of corrosion products. However the pit was observed in isolation within a silvery-gold coloured surface. The pit itself had a bright metallic appearance, also silvery-gold in colour, and measured 1.75mm in diameter and approximately 1.5mm in depth (see Inset A on Figure 6.81). A series of “bench marks” stepped down into the pit centre at roughly equal spacings and shape, all of which exhibited the same bright, metallic surface as its surrounds.

Figure 6.82a shows an SEM surface image, in which the pit steps and bright, metallic surface were clearly defined. The step pattern also continued on a finer scale outside of pit boundary. A close-up image of the base of the pit (at Point A) is shown in Figure 6.82b, which shows an irregular, crystalline surface encircled by a series of brittle cracks. An EDS analysis revealed a high count of iron, with smaller levels of chlorine (Figure 6.82c).

<table>
<thead>
<tr>
<th>Phase Priority</th>
<th>Compound Identified</th>
<th>Chemical Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main</td>
<td>Iron</td>
<td>Fe</td>
</tr>
<tr>
<td>Semi-dominant</td>
<td>Magnetite</td>
<td>Fe$_3$O$_4$</td>
</tr>
<tr>
<td></td>
<td>Goethite</td>
<td>$\alpha$-FeOOH</td>
</tr>
<tr>
<td>Possible Minor Phase</td>
<td>Xitieshanite</td>
<td>Fe(SO$_4$)(OH)·6H$_2$O</td>
</tr>
</tbody>
</table>

Table 6.5: XRD analysis of bright, metallic surface from Figure 6.77b
6.4 Other Observations

This scan, however, encountered a high degree of background noise and a low count, which may indicate the possible presence of an amorphous substance of poor orientation of the sample.

![Image of bright, metallic pit with concentric rings or steps](image)

Figure 6.81: An example of a bright, metallic pit with concentric rings or steps

![SEM image](image)
![Close-up of SEM image at Point A](image)
![EDS scan](image)

(a) SEM image  
(b) Close-up of SEM image at Point A, Figure 6.82a

Figure 6.82: Circular, stepped pit observed on shear ligature from Beam 118
6.4 Other Observations

6.4.3 Microbiological tests

As discussed in Chapter 3, whilst the presence of microbiological organisms in the corrosion process of reinforced concrete may be unlikely, it cannot be discounted. Due to the observation of wet, black rusts (Section 6.3.4.3) and bright, metallic pit surfaces (Section 6.4.2), representative samples were analysed for Sulphate Reducing Bacteria (SRB) and Iron Related Bacteria (IRB).

Samples were analysed using ready-made culturing test kits. For the detection of SRB, the Biosan Laboratories “Sani-Check” test kit was used. The Droycon Bioconcepts Inc. “BART-IRB” test kit was used to detect the presence of IRB. Surfaces were either collected using sterile instrumentation or swabbed (Figure 6.83). The samples selected for analysis were prestres

![Figure 6.83: Taking a swab sample from a steel surface for microbiological testing](image)

In summary, of the 12 samples tested for SRB only two test results showed an indication of bacterial activity. The response from the positive tests were weak, with results taking over a week to react which indicates a low to negligible presence of SRB (according to the SRB test-kit documentation). Figure 6.84 shows some representative samples that have been tested for SRB.

Results for samples tested for IRB were opposite to that observed for the SRB samples. Of the 11 sites that were tested, 100% of the samples indicated that iron-related bacteria were likely to be present, in particular iron oxidising bacteria. Figure 6.85 shows a selection of
samples showing positive results for IRB. These results were compared with the test-kit guide for the determination of the dominant bacteria. The majority of cases appeared to subscribe to the “BR” or “BC” category, indicating IRB. In one sample, air bubbles were present at the top of the test, as shown in Figure 6.85b. According to the test-kit guide, this result may be an indication of anaerobic bacteria.

![Figure 6.84: Example of SRB tests for steel samples](image)

![Figure 6.85: Example of IRB tests for steel & concrete samples](image)
6.5 Summary

In summary of these observations, both general and pitting corrosion was observed on both conventional reinforcement and prestressing strands from all three beams. Complete section losses were observed on shear ligatures and longitudinal reinforcement adjacent to crack zones. Complete section losses were also registered for prestressing strands from both tendons in Beams 17/3 and 118. Beam 17/3 was found to have the worst corrosion in relation to cross-sectional area losses. Beam 17/4, whilst in the best condition, had prestressing strands that exhibited significant area section losses despite there being no visible evidence from the concrete surface. A wide variety of corrosion products were found on both conventional reinforcement and prestressing strands, which included Magnetite, Wustite, Lepidocrocite and Akaganeite. Unusual corrosion products were also detected, such as the chlorine-based Green Rust (I) and Iron (III) Oxy- and Hydroxyl- Chlorides. Other unusual observations included the formation of aggregate rims, iron chloride “weeping”, wet and dry “black rusts”, and bright metallic surfaces on steel surfaces and in the base of pits. Tests for the presence of microbiological organisms were positive in some areas. These observations are discussed further in Chapter 7 in relation to results from Chapters 4 and 5 to review the degree of correlation between field investigations and the physical condition of the steel.
Chapter 7

Discussion

7.1 Introduction

A number of diagnostic tests and detailed investigations have been conducted on three beams from the old Sorell Causeway Bridge, including those carried out whilst the bridge was in service. These results have been discussed independently in Chapters 2, 4 to 6. However, what is yet to be discussed is the correlation of these results to one another, in particular the comparison of pre-demolition data to the physical condition of the prestressing strands and conventional reinforcement determined after demolition. This chapter integrates these pre and post-demolition observations and conclusions to gain insight into firstly how corrosion affects the overall performance of a prestressed concrete member and secondly the accuracy of the pre-demolition observations in regards to its condition and load capacity. Additional comment is provided on other observations made during the course of this project.

7.2 Correlation between Visible Observations and Steel Condition

7.2.1 Visual Observations

Visual inspections of reinforced concrete structures represent the initial point of reference in the determination of the condition of a structure. In practice, inspectors rely on visual keys such as corrosion-related defects to draw to attention the structural members requiring further investigation and maintenance. Thus a question regarding the degree of
correlation between visual surface defects to the physical condition of the steel is raised. Based on the visual observations made on each beam for the present study and the previous investigations conducted prior to the Sorell bridge being decommissioned, it is not unreasonable to assume that Beams 17/3 and 118 were in the worst condition. Indeed, the condition of the steel for Beam 17/3 appeared to confirm this assumption, with large cross-sectional area losses on the lower tendon prestressing steel for almost the full length of Bay 5.

In relation to Beam 17/4, however, attention was predominantly focussed on instances of spalling and shear ligature corrosion on Bay 5 in lieu of the condition of the prestressing steel. As was noted in Chapter 6, some regions of the prestressing steel had suffered 25% cross-sectional area losses and yet this was not evident from the beam surface. Similarly, the severity of prestressing strand section loss in Beam 118 was not fully realised by observing the longitudinal crack widths. And in all instances where pitting had occurred, there was no visible evidence of this corrosion on the concrete surface.

### 7.2.2 Rust Staining and Steel condition

A key indicator for corrosion-related problems on reinforced concrete structures is the presence of rust staining on the surface of the concrete. However, limited rust staining was evident on all three test beams, especially Beams 17/3 and 118. Upon closer inspection, the shear ligatures and longitudinal rebars were responsible for the observed rust staining, with very little or no evidence from the prestressing strands (see Chapter 6). This latter observation and the fact that severe cross-sectional area losses were measured on strands from both Beams 17/3 and 118, this begs the question: where has the corrosion product gone?

It was noted in Chapter 6 that corrosion product has built up in voids present in the tendons, which may explain the lack of rust staining. However, assuming the volume of these corrosion products is 3 - 6 times greater than the base metal itself and complete section losses had occurred on strands in some locations, it is unlikely that these voids were large enough to accommodate the resulting corrosion products without some visible
evidence.

Observations in Chapter 6 have discussed the presence of ferrous chloride weeping out of steel/concrete interfaces. Ferrous chloride is known to form under pitting conditions, where low oxygen concentrations and low pH exist (see Chapter 3). It is postulated that some of the iron from the strand corrosion process had remained in solution, however there is no conclusive evidence to support this statement. The abundance of chloride ions and the limited oxygen available may have assisted in the formation of ferrous chloride. It is further hypothesised that the ferrous chlorides were either washed away (due to tidal splash) or merely leached out of the concrete (which has not previously been identified in field investigations, perhaps due to the transparency of the liquid). It is therefore recommended that further studies be conducted on the role of ferrous chlorides and the quantification of corrosion products in reinforced and prestressed concrete.

7.2.3 Crack Widths and Steel Condition

As previously discussed in Chapter 4, visual observations provide the inspector with first impressions of a structure, which subsequently lead to an initial appraisal of its condition. For the inspectors conducting the condition assessments of the Sorell bridge, the observation of the severe defects observed on Beam #12 of Span 17 (Beam 17/3) must have presented some alarm but due to the distinct lack of rust staining, the root cause of the cracking was not automatically blamed on corrosion. After all, this beam was situated directly in line with traffic wheel paths and as such structural deficiencies could not be ruled out. It was not until Infratech’s 1999 report [157] that it was suggested more specifically that the longitudinal web cracking was most likely due to corrosion of the strands, and that section losses in these areas were probable. After the demolition of Beams 17/3 and 118, it was confirmed that the longitudinal web cracking did follow the trajectory of the tendons and that corrosion was observed on strands in areas adjacent to cracking.

The amount of corrosion observed across both prestressing strands and conventional reinforcement was discussed in Chapter 5, where it was concluded that section losses were most significant in areas that exhibited cracking or spalling. This discovery in itself is not new; conventional literature states that where cover cracking occurs in reinforced concrete, accelerated corrosion and greater section losses are likely to be observed [50, 57, 63, 150, 251].
To discuss this finding further, Figure 7.1 show the comparison of section losses in relation to measured crack widths for Beams 17/3 and 118. Generally speaking, where corrosion losses were greater than 50%, crack widths were greatest. However, where the most significant corrosion losses were recorded, this did not automatically translate to the greatest crack widths. Take Beam 17/3 for example (Figure 7.1a); both Face C and D had crack width maximums of approximately 20mm with cross-sectional area losses averaging approximately 55% at these locations. However, the worst section losses of approximately 80% were recorded adjacent to crack widths of 10 - 15mm. Despite these differences, it is reasonable to assume that for a crack width of 10mm or more that significant corrosion should be anticipated.

In comparison, Beam 118 exhibited maximum crack widths approaching 10mm (Figure 7.1b). This was observed in a section of spalling on Face D that did not directly correspond with the trajectory of the tendons. Indeed, cross-sectional area losses in the strands located in the direct vicinity were not significant. Where large steel area losses are recorded, crack widths did not exceed 5mm. It thus appears that crack widths may not always be a reliable indicator of corrosion severity. Perhaps the occurrence of severe and widespread spalling on Face D of this beam affords a better indication of corrosion for this instance.
7.2 Visual Observations & Steel Condition

Figure 7.1: Correlation between crack width and prestressing steel condition
7.2 Visual Observations & Steel Condition

Figure 7.1: Correlation between crack width and prestressing steel condition (cont'd)
7.2.4 Summary of Findings

The following key points regarding visual observations determined for the present investigation are summarised as follows:

- Cracking generally coincided with the trajectory of the prestressing strands, which appears to have been instigated via general corrosion of the strands
- Evidence of significant corrosion was not always visible at the concrete surface (for example Beam 17/4)
- Instances of severe pitting across the conventional reinforcement were not detectable via visual inspection
- There was limited evidence of rust staining at the concrete surface. The majority of rust staining appeared to stem from conventional reinforcement, with limited or no corrosion staining from the prestressing strands
- Some correlation was found between crack widths and severity of corrosion of the strands, however the largest widths did not always signify the location of the strands with the greatest cross-sectional area losses

7.3 Non-Destructive Test Outcomes Relative to Steel Condition

As discussed in Chapter 4, many techniques have been developed over the years in response to demands from asset managers to know with some certainty the condition of a reinforced concrete structure showing symptoms of corrosion. For this project, particular interest has focussed on utilising the half-cell potential and concrete resistivity methods, as these are the most inexpensive and thus commonly used techniques. These were also the predominant NDT methods employed in assessing the condition of the Sorell Causeway Bridge. However, do results from these techniques subscribe to the published “corrosion” limits and do they provide an accurate indication of the risks posed by corrosion? The following paragraphs provide comment on the degree of correlation between the NDT results obtained for the present investigation and the physical condition of the steel.
7.3 Non-Destructive Testing & Steel Condition

7.3.1 Half-Cell Potentials

7.3.1.1 General Observations

Half-cell results obtained during the condition assessment conducted by Façade in 1994 [118] showed that increasingly negative potentials were found towards the base of the beams and that corrosion was likely on one beam due to the identification of ‘significant areas’ with potentials more negative than -350mV (in relation to a Copper-Copper Sulphate (CSE) reference electrode); however this conclusion was not made for Beam 17/3. The report showed half-cell results for the condition of the conventional reinforcement, in particular the shear ligatures, and no further tests had been carried out on the post-tensioning strands although this is not specifically stated. The inspectors claimed that a strong degree of correlation was found between the half-cell potentials measured and the steel condition, stating that, “significantly more negative readings were obtained at visibly corroded and spalled areas...3 out of 4 cases showed a correlation between corrosion and higher negative readings.”

To verify this statement, complete half-cell potential surveys in relation to conventional reinforcement were conducted for all faces of Beams 17/4, 17/3 and 118 using a copper/copper sulphate reference electrode. All information was reviewed based on the corrosion risk limits provided in the ASTM C876 guidelines [43], where corrosion is more likely when potentials were more negative than -350mV CSE. Results were subjected to additional interpretations in accordance to recommendations provided by Chess and Gronvold [93], Figg and Marsden [122], and Vassie [302]. Results are shown in Chapter 4.

In general, it was found that corrosion potentials became more negative towards the base of the beams and adjacent to visible surface defects as determined by Façade. However only relatively small areas were identified as having a high likelihood of corrosion. How this relates to the condition of the steel is now discussed.

In reviewing equipotential contours for the entirety of each beam, it appears that the plots narrowed the field of view in relation to areas of corrosion, such as Bay 5 on all beams. However, in relation to the ASTM C876 limits [43] and more specifically the condition of the steel, inconsistencies were observed. Figure 7.2 shows the half-cell potential survey results in relation to the physical condition of the reinforcing bars for each test beam.
7.3 Non-Destructive Testing & Steel Condition

Reviewing results purely based on their absolute value, these plots showed that whilst some areas of corrosion on the shear ligatures were identified by the surveys (denoted by zones more negative than -350mV, see Point A on Figures 7.2b and 7.2c respectively), there are many more that were not (see Point B on Figures 7.2a, 7.2b, and 7.2c respectively). Zones of severe corrosion appeared to lie between potential contours of -50 to -500mV, which is an extremely wide range. Based on these observations, it appears that care should be employed when interpreting absolute potentials for likely corrosion zones when using the ASTM C876 limits [43].

Alternatively, areas of extremely negative potentials were measured in zones where little or no corrosion had occurred. A good example is shown along the soffit of Beam 17/3 in Figure 7.2b. Here, absolute potentials as negative as -600mV were identified; according to ASTM C876 limits [43], corrosion was almost a certainty. Indeed, due to the highly negative value corrosion was anticipated to be severe with evidence of the corrosion visible on the concrete surface (such as spalling or rust staining) [66, 259, 288]. However, the majority of the rebars were found in a passive state, with only small, isolated areas showing slight corrosion. The opposite case is true for the soffit along Beam 118, with severe corrosion observed on rebars existing within a potential range of between -150 to 200mV (Figure 7.2c).

Potential contour patterns containing localised minima and potential gradients were reviewed in relation to the steel condition, as recommended in the literature [93, 122, 302]. Making allowances for some graphical distortions and assumptions made during the generation of the automated contouring plots, a small improvement was found in identifying areas at risk of corrosion. Point A on Figure 7.2a show examples where the position of the local minima coincided with the corrosion of the ligatures (despite a lower potential boundary of -250mV). Steeper gradients at Point B also showed compliance with the recommendations for gradient interpretation. However, not all minima and steep gradients translated to the identification of corrosion zones. Therefore it appears that the interpretations recommended in the literature appear to be inconsistent with potentials obtained for the current investigation.
7.3 Non-Destructive Testing & Steel Condition

Figure 7.2: Correlation between half-cell potential survey and conventional reinforcement condition
Figure 7.2: Correlation between half-cell potential survey and conventional reinforcement condition (cont'd)
7.3 Non-Destructive Testing & Steel Condition

Figure 7.2: Correlation between half-cell potential survey and conventional reinforcement condition (cont'd)
Attention is now turned to the validity of the half-cell potential survey in detecting areas of corrosion across the prestressing strands. As mentioned in Chapter 4, measuring the half-cell potential of prestressing strands is not normally adopted on site due to the inaccessibility of the strands; however the demolition of the beams for the present investigation was used for this advantage. Variabilities similar to those discussed in the previous paragraphs were observed in the correlation of potential values to the condition of the prestressing steel. Beam 118 exhibited the greatest degree of compliance with the ASTM C876 limits \cite{43}, with the location of corroding prestressing strands within zones more negative than -350mV. Where severe corrosion existed, there was a trend for decreasing potentials between -400 and -450mV (denoted by Point A on Figure 7.3a). However, the same statement could not be made for Beams 17/3 and 118. For Beam 17/3, strand section losses ranged between 10 and 50\% over a potential range between -50 and -400mV (not necessarily in that order). Contour gradients and the location of minima were also found to contradict the literature recommendations.

It has already been observed that highly negative potentials existed along the soffit of Beam 17/3 without any apparent correlation to the condition of the conventional reinforcement (and vice versa for Beam 118). However, observing the half-cell potentials at the same location on Beam 17/3 in relation to the condition of the prestressing strands (Figure 7.3b), there appeared to be a better degree of correlation. This raises the issue of the significance of electrical connections. As discussed in Chapter 4, similarities were observed between equipotential plots for varying connection points. Ohta et al \cite{217} has noted similar findings, with almost identical half-cell patterns between the reinforcing bar and prestressing sheath connections; however no explanation is offered by the authors as to why this may be the case. It is thus postulated that half-cell tests may have the capability of identifying corrosion zones regardless of its electrical connection. It may also mean that equipotential plots can be influenced by geometric constraints or the proximity of steel layers. It is recommended that further investigations be conducted to clarify these observations.
Figure 7.3: Correlation between half-cell potential survey and prestressing strand condition (lower tendon)
Figure 7.3: Correlation between half-cell potential survey and prestressing strand condition (lower tendon) (cont'd)
7.3 Non-Destructive Testing & Steel Condition

7.3.1.2 Half-Cell Potentials and Pitting Corrosion

One of the original concerns surrounding the bridge beams was the unknown threat of pitting corrosion. Vassie [302] states that steel affected by pitting within zones containing high levels of chlorides should be identifiable on the equipotential plots. Bertolini et al [57] states that chloride-induced pitting is normally indicated by potentials ranging between -400 to -700 mV CSE. Bungey and Millard [75] have also stated that reducing grid measuring points to 100mm squares will facilitate in the detection of microcells or pitting. Therefore it was thought beneficial to cross-reference instances of pitting or localised corrosion with the potentials measured for each beam. As noted in Chapter 6 the corrosion of the prestressing strands can predominantly be classified as general corrosion and that the most severe cases of pitting/localised corrosion were found on conventional reinforcement, especially those adjacent to crack zones. Typically, the majority of pits noted across the rebars were incorporated into areas of general section losses and as such it was anticipated that the half-cell potentials would identify these areas in lieu of the pitting. However, two specific examples of significant isolated pitting is shown in Figure 7.4.

Figure 7.4a shows a shear ligature from Beam 17/4 which had an isolated, elongated pit. No external signs of this pit existed on the concrete surface. The equipotential contours envelop this area between -200 and -250 mV, which according to the ASTM C876 limits [43] correlates to an uncertain probability of corrosion. Referring to the existence of localised minima and contour gradients, there are no indications whatsoever adjacent to this pit. Thus the plot is ambiguous at best and to those reviewing such data this pit would go undetected. Figure 7.4b shows a similar case, where an isolated pit is observed within a more substantial corrosion zone. The pit itself lies within an elongated maxima of between -250 and -300 mV; as such there is nothing in the literature to suggest that a maxima can be indicative of corrosion activity. As such, the use of equipotential plots appears to be inconclusive in relation to pitting.

As an additional comment, it may be that grid spacing of 100 mm was insufficient to detect localised corrosion for this application, and perhaps more instances may have been detected if the grid had been smaller. Pitting may also have been easier to detect if bar sizes had been larger; the size of 6 mm shear ligatures is very small (and now redundant)
in comparison to general commercial practice. In response to these findings, however, the practical nature of the test is brought into question. Reducing the grid size may assist in detecting areas of localised corrosion, however the method is time consuming and as discussed in the previous section, not always accurate. Thus, based on the above observations, it appears that the use of the half-cell potential technique is more suited for the identification of whole corrosion zones attributable to general corrosion.
7.3.1.3 Summary of Findings

In summarising these discussions, the following points are made:

- it appears that the use of half-cell potentials offers a technique that can generally detect zones where corrosion is likely to be occurring, however this should be applied holistically

- Minimal correlation was found between the steel condition (such as initiation and severity) and absolute half-cell potential limits as outlined by ASTM C876 [43]

- Interpretation of equipotential contour gradients and localised minima did not appear to assist in detecting areas of corrosion (as recommended in the literature [93, 122, 244, 302])

- Half-cell potentials more negative than -500mV did not appear to represent an upper limit where corrosion is automatically visible on the concrete surface, if indeed it represents corrosion at all (as recommended in the literature [66, 259, 288])

- Pitting was not detected by the presence of local minima and potential gradients on equipotential contour plots using a grid spacing of 100mm (as recommended in the literature [75])

- Some degree of similarity was observed on equipotential contour plots between steel connections that are electrically isolated; geometric constraints and the proximity of steel may also have influenced the results.
7.3 Non-Destructive Testing & Steel Condition

7.3.2 Concrete Resistivity

Concrete resistivity readings of the beams insitu taken by Façade Engineering \[118\] were extremely sparse and inconclusive (Chapter 2). However of the results that were obtained showed that the resistivity was very low (less than 2kΩ.cm), suggesting that the corrosion rate was likely to be very high (see Table 4.9 \[69\]). Reasons given for the low values included “insufficient electrode penetration” and “low resistivity chloride surfaces”. Subsequent resistivity results determined for this thesis reveal a different account of the resistive characteristics of the concrete, as was shown in Chapter 4. These results are now discussed in relation to the steel condition.

Figure 7.5 shows recreated equiresistance plots for each beam shown in Chapter 4 in relation to the condition of the conventional reinforcement (results shown are those obtained with the MEGGER\textsuperscript{®} Meter). There is a trend for resistivities to decrease towards the base of the beam and Diaphragm (iv) for Beams 17/3 and 118, which is where the most significant corrosion is observed. Indeed, resistivities as low as 9kΩ.cm were found adjacent to some of the worst section losses. Note on Figure 7.5c the extent of corrosion on shear ligatures adjacent to Diaphragm (iv) on Face D, which coincided with lower resistivities in comparison to the remainder of the bay. In some instances, corrosion appeared to be contained within lower resistivity areas, with zones showing high resistivities between 50 and 100kΩ.cm overlaying zones showing no evidence of corrosion initiation.

Figure 7.6 shows the same resistivity data superimposed onto the prestressing steel condition. The results show somewhat similar trends to those noted above, however there was a reduced degree of correlation. For example, the lower tendon in Beam 17/3 at Point A on Figure 7.6b shows strand cross-sectional area losses that were approaching 80% adjacent to areas of resistivity averaging 15kΩ.cm. Similar examples were observed on Face D of Beam 118, with a trend of decreasing resistivities less than 20kΩ.cm overlay significant levels of corrosion for both upper and lower tendons (see Point A on Figure 7.6c). In comparison however, it was noted for Beam 17/3 that section losses of approximately 50% existed within a higher resistivity environment of approximately 50kΩ.cm (see Point B on Figure 7.6b), with similar findings for Beam 118 (see Point B on Figure 7.6c). It is this latter observation that requires further discussion.
7.3 Non-Destructive Testing & Steel Condition

Figure 7.5: Correlation between concrete resistivity and conventional reinforcement condition

(a) Beam 17/4

Figure 7.5: Correlation between concrete resistivity and conventional reinforcement condition
Figure 7.5: Correlation between concrete resistivity and conventional reinforcement condition (cont’d)
7.3 Non-Destructive Testing & Steel Condition

Figure 7.5: Correlation between concrete resistivity and conventional reinforcement condition (cont'd)
7.3 Non-Destructive Testing & Steel Condition

Figure 7.6: Correlation between concrete resistivity and prestressing strand condition

(a) Beam 17/4
Figure 7.6: Correlation between concrete resistivity and prestressing strand condition (cont'd)
7.3 Non-Destructive Testing & Steel Condition

Figure 7.6: Correlation between concrete resistivity and prestressing strand condition (cont'd)
Evidence of corrosion of steel occurs at varying levels of resistivity. Bearing in mind that resistivity measurements only provide an indication of the ability of the concrete to facilitate the corrosion process, these observations should come as no surprise. However, as resistivity measurements provide an indication of the rate of corrosion, measurements obtained have been compared to the corrosion rates estimated from steel cross-sectional area losses. The empirical resistivity limits discussed in Table 4.9 (from Chapter 4) have been modified to reflect the corrosion rates outlined by Bertolini et al [57], and are shown in Table 7.1. A “very high” corrosion rate for steel embedded in chloride contaminated concrete relates to a corrosion rate of greater than 100μm/yr, which equates to a resistivity level of less than 5kΩ.cm.

<table>
<thead>
<tr>
<th>Resistivity (kΩ.cm)</th>
<th>Empirical Corrosion Rate [69]</th>
<th>Estimated Corrosion Rate μm/yr [57]</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 5</td>
<td>Very high</td>
<td>≥ 100</td>
</tr>
<tr>
<td>5 - 10</td>
<td>High</td>
<td>10 - 100</td>
</tr>
<tr>
<td>10 - 20</td>
<td>Low</td>
<td>2 - 10</td>
</tr>
<tr>
<td>≥ 20</td>
<td>Negligible; Concrete too dry</td>
<td>≤ 2</td>
</tr>
</tbody>
</table>

Corrosion rates for strands from the lower tendon of Beams 17/3 and 118 were estimated to be greater than 100μm/yr (see Appendix C), a very high rate (based on steel observations made in locations where the worst section losses were measured). With the exception of one result obtained for Beam 118 using the MEGGER® meter, no resistivities less than 5kΩ.cm were recorded, therefore casting doubt over the resistivity limit recommendations. Of the 5% of results that were less than 10kΩ.cm, the majority of these occurred on Beam 118 and were found along the base flange or over zones of spalling and not adjacent to areas of significant strand corrosion. Of the resistivity readings adjacent to the latter sections, these ranged between 11 and 35kΩ.cm indicating a corrosion rate of anywhere between 2 and 100μm/yr. This is quite a range considering that the estimated corrosion rate in this area was likely to be 100μm/yr.

Attention is now turned to the severity of corrosion observed in relation to resistivity values, as there were varying resistivity readings observed adjacent to similar corrosion
7.3 Non-Destructive Testing & Steel Condition

conditions. For example, where corrosion losses were severe (i.e. denoted as ‘black’ zones on all condition diagrams), resistivities ranged between 30 to 76kΩ.cm for Beam 17/3 (Figure 7.6b) and 16 to 47kΩ.cm for Beam 118 (Figure 7.6c). Resistivities as low as 12kΩ.cm existed adjacent to areas showing less than 10% section losses. Alternatively, resistivities greater than 30kΩ.cm were consistently measured over tendons from Beam 17/4 also showing section losses similar to those observed in the latter example. According to the threshold limits in Table 7.1, this corresponds to a negligible corrosion rate, which from the outset appears to correlate reasonably well to the physical condition of the steel. Similar observations are made between resistivity results and the condition of the conventional reinforcement. Therefore, whilst there appears to be a general trend of low resistivities towards corrosion zones, high variability still exists amongst the results and the recommended corrosion rate limits do not appear to comply with the estimated corrosion rates of the steel.

Of course, the above discussions assume that the resistivity results obtained are completely reliable and that error factors outlined in Chapter 4 either do not apply or are insignificant. The most critical factors applying to this project are the presence of highly resistive surfaces layers (attributable to carbonation), boundary effects from geometric limitations and the presence of reinforcing steel (both conventional and prestressing). Due to the information presented in Chapters 4 and 5, it is known that errors due to these effects are likely to be unavoidable. For example, the depth of carbonation and the close spacing of reinforcement is likely to present issues, however due to the fixity of the electrode spacing corrections will be required. The methods for making such corrections is ambiguous. Whilst Millard and Gowers [141, 198, 201] have presented very thorough data on factors affecting resistivity measurements, these recommendations are based on finite element modelling to which only a limited number of scenarios are presented. There are also no recommendations on how to apply such corrections, even though RILEM support their implementation [243]. It is thus difficult to make corrections on results obtained for this project based on their data. It is, however, estimated that the true resistivity could be in the order of between half (due to boundary effects) to two times (due to steel effects) the measured resistivity.
7.3.2.1 Summary of Findings

In summarising these discussions and the findings from Chapters 4 and 5, the following points are made:

- The resistivity measurements obtained for this thesis do not correlate well to results obtained in the 1994 Façade Engineering report [118]
- The lowest resistivity measurements were recorded on Beam 118, which appeared to show a greater area of bars with corrosion; Beam 17/4 recorded the highest resistivities
- Measured resistivities did not always appear to subscribe to the empirical resistivity limits regarding corrosion rates when compared to the actual condition of the steel [69, 89, 138, 243]
- Low resistivity measurements did not always correlate to areas of corrosion on both prestressing and conventional steel
- It is likely that carbonation, geometric constraints and steel locations have influenced measured resistivities by a factor ranging from 0.5 and 2 times [141, 198, 201]

7.4 Steel Condition in Relation to Chloride Levels and Depth of Carbonation

7.4.1 General Observations

It is generally thought that with increasing chloride levels, a subsequent increase in the frequency of steel corrosion should occur [57, 131, 298]. Due to the Sorell Causeway Bridge being located in an aggressive marine environment and the inclusion of the additive calcium chloride (CaCl₂) into the beams, the testing for chloride levels had been a predominant concern for DIER. As such, the most detailed investigations prior to the Sorell bridge being decommissioned (with the exception of load testing) was chloride profiling. As a matter of course, carbonation depths were also assessed. The conclusions from both the 1994 Façade [118] and 1997 Taywood Engineering reports [277, 278] were that the chloride levels across the beams exceeded the recommended chloride threshold of 0.06% Cl⁻ (by weight of concrete), with Taywood Engineering stating that “these results
clearly suggest that there is widespread corrosion initiation associated with both the ligatures and the strands.” There was disagreement between the two reports regarding the depths of carbonation, with Façade finding maximum carbonation depths in the beams of 19mm, whereas Taywood Engineering found negligible depths.

Additionally, more detailed chloride and carbonation profiles were carried out as part of the current study, of which greater than 70% of chloride samples were more than 0.4% $\text{Cl}^-$ (cement) (where the cement content averaged 21% $\text{Cl}^-$ by weight of concrete). The elevated chloride levels may be attributable to the presence of calcium chloride ($\text{CaCl}_2$), however the exact proportion attributable to this additive is unknown (indeed, this additive would not have been officially identified if it were not for the letter received from John Holland Constructions in 1976 [98]). It is thought that these levels could range between 0.5 and 2% [13, 204, 230]. The literature states that chlorides from the additive will most likely be bound within the concrete, with only free chlorides that have ingressed from the external environment participating in the corrosion process. However recent works have identified that with the event of carbonation the bound chlorides may be loosed and also contribute (see Chapter 5 for further discussion). Therefore, all chloride concentrations adjacent to reinforcement have been reviewed in conjunction with carbonation levels.

Figures 7.7 and 7.8 show data for chloride and carbonation levels adjacent to conventional and prestressing steels, of which the physical condition of the steel is also shown. In short, limited correlation was found between the steel condition and the chloride levels for this project, and the assumption of severe corrosion occurring within high chloride concentration zones does not appear to be substantiated. Several representative examples will now be discussed. A number of examples were observed where high chloride levels did equate to significant corrosion, which is denoted by Point A on each subfigure from Figures 7.7 and 7.8. Point A on Face C from Beam 17/4 (Figure 7.7a) shows an example where a chloride concentration adjacent to both the shear ligature and prestressing strands was 2.6% $\text{Cl}^-$ (cement). At this location, both sets of steel exhibit isolated and severe corrosion (corrosion on the shear ligature is shown in Figure 7.4a). Carbonation depths vary for each example (ranging from negligible to significant depths of approximately 30mm).
Alternatively, high chloride levels were also associated with little or no corrosion on the steel. This is shown by Point B on Figures 7.7 and 7.8. Specifically, at Point B on Face D from Beam 17/3 (Figure 7.7b), a chloride concentration of 1.2% $Cl^-$ (cement) was measured adjacent to this shear ligature and lies within carbonated concrete. However, the steel at this location remained passive, with no evidence of corrosion initiation. Interestingly, the prestressing steel in the same location was undergoing severe corrosion at lower chloride levels (0.9% $Cl^-$ (cement), see Figure 7.8a).

Perhaps more concerning are the instances of low chloride levels measured adjacent to zones of significant corrosion. These results are highlighted by Point C on Figures 7.7b and 7.8b. Referring to the same example from Beam 17/3 discussed in the previous paragraph, severe and localised corrosion was registered on the shear ligature at Point C; significant section losses were also noted for the prestressing strands at this location. However, a chloride level of 0% $Cl^-$ (cement) was been recorded at both steel levels. Carbonation depths at this point were not obtained, however it is thought that levels may range between 10 and 30mm. Errors may be present in this chloride reading (due to a small sample size) however samples of similar size do register some level of chlorides.

It can be said with some certainty that the presence of chlorides has had some involvement with the corrosion of the shear ligatures and longitudinal rebars, due to the aggressive and localised nature of the section losses observed. This is supported by analysis conducted by XRD and SEM on products obtained from corrosion sites, which confirmed the presence of chloride ions. The source of the chlorides cannot be confirmed with the tests undertaken during the present investigation, meaning that the corrosion process cannot be directly attributed to the additive of calcium chloride. The ingress of chlorides was highly likely for the Sorell bridge, due to its location in an aggressive marine environment. Indeed, high levels are anticipated due to the beams being constantly affected by tidal splash (where high surface chloride levels and deeper chloride penetration can exist [57]). However, the permeability of the concrete (which for Beams 17/4 and 17/3 had fairly low permeabilities, as discussed in Chapter 5) and the highly resistive surface layers due to carbonation may thus limit the diffusion of chlorides.
7.4 Chlorides, Carbonation & Steel Condition

(a) Bay 5 of Beam 17/4

Figure 7.7: Steel condition with respect to carbonation and chloride levels
7.4 Chlorides, Carbonation & Steel Condition

Figure 7.7: Steel condition with respect to carbonation and chloride levels (cont’d)
7.4 Chlorides, Carbonation & Steel Condition

Figure 7.7: Steel condition with respect to carbonation and chloride levels (cont’d)
7.4 Chlorides, Carbonation & Steel Condition

Figure 7.8: Prestressing steel condition with respect to carbonation and chloride levels

(a) Bay 5 of Beam 17/4
7.4 Chlorides, Carbonation & Steel Condition

Figure 7.8: Prestressing steel condition with respect to carbonation and chloride levels (cont'd)

(b) Bay 5 of Beam 17/3

[Diagram showing carbonation and chloride levels with corresponding conditions]

- No corrosion, good condition
- Active corrosion, $A_e < 30$
- Moderate corrosion, $30 < A_e < 60$
- Significant corrosion, $60 < A_e < 90$
- Severe corrosion, $A_e < 90$
- Rust staining, no section loss

Longitudinal Sample × Cross-Sectional Sample

Carbonation Depth (mm)

0 5 10 15 20 25 30 35 40

D(v) D(iv) D(v) D(iv)
7.4 Chlorides, Carbonation & Steel Condition

Figure 7.8: Prestressing steel condition with respect to carbonation and chloride levels (cont'd)
It is accepted that not all chlorides will be bound during cement hydration and as such a small percentage may remain as free chlorides (see Chapter 5). Further chlorides may be liberated from the cement matrix in the event of carbonation adjacent to reinforcement, thus increasing the percentage of chloride ions participating in the corrosion process. Due to the irregularity of chloride profiles measured (disallowing the hypothesis of a base chloride level), the unknown original quantity of calcium chloride added during casting, the lack of analytical evidence of Friedel’s Salt (which facilitates chloride binding), and the amount of free chlorides remaining after binding, evidence of chloride binding and the exact percentage of chlorides released from binding remains uncertain.

Concerning the prestressing strands, it could not initially be assumed that chlorides had participated in the corrosion process. This is due to the measurement of minimal chloride levels in the grout (thus confirming the statement by Cook that calcium chloride was not added to the tendons [98], see Chapter 4) and the observation of general corrosion in lieu of the aggressive pitting observed on reinforcement. The general corrosion may be attributable to small voids present at the tops of the tendons (see Chapter 6), where the alignment of all strand groups after prestressing were pressed up against the top of the tendon leaving little room for grout to penetrate into this small gap and between the strands (or now non-existent due to the evaporation of bleed water from the grout). Within this zone, the pH was found to range between 7 and 9. At this pH level, corrosion may initiate and accumulate corrosion products in the void. However, the analysis of these corrosion products show a prevalence of chloride-based rusts (such as akaganeite and GR(I)) and as such the participation of chlorides in the corrosion process cannot be excluded. The predominant source of these chlorides is assumed to be the calcium chloride, which is likely to have been present at the tendon perimeter and subsequently released with a drop in pH. As for the general corrosion profile, it has been noted in the literature that consistently high chloride levels can obscure the effects of pitting, giving the steel the appearance of general corrosion [57, 67, 285]. As such, localised section losses and elongated pits were observed in isolation across the steel in all beams, therefore it can be concluded that chlorides have participated in the corrosion process.

In summary of these observations and those made in Chapter 4, chloride levels determined for this thesis generally agree with those found in the 1994 Façade and 1997 Taywood...
Engineering reports [118, 277, 278] with irregular and elevated chloride levels for various locations. Chloride levels did not appear to subscribe to the normal diffusion model of decreasing chlorides with increasing depth [67, 163, 230].

In relation to the chloride results and the condition of the steel, the correlation between high chloride levels and corrosion activity (and subsequently corrosion initiation) appears to be weak. Active corrosion was noted to occur at chloride levels as low as 0.3% $Cl^-$ (cement) (excluding the 0% $Cl^-$ (cement) recorded on Beam 17/3) but passive states were also observed on steel with a maximum chloride level of 1.8% $Cl^-$ (cement). These observations are not isolated; for example, Schupack and O’Neil [254] states in a similar demolition study that it was “a surprising fact that with this amount of chloride contamination [exceeding the recommended limit]...that the...33 year old beams show limited evidence of web-stirrup corrosion.” Bruce et al [71] notes elevated chloride levels across the Hamanatua Bridge but observes that no prestressing strands in this vicinity are corroding. Thus, using high chloride levels to predict corrosion initiation (as is often done in practice) appears to be unreliable.

7.4.2 Additional Discussion on Carbonation Effects

With regards to carbonation depths, it was surprising to see the depths observed across all three beams considering the bridge is classified to be located in an aggressive marine environment subjected to tidal splash, with average maximum depths of 30mm recorded
(excluded complete carbonation of the web adjacent to Diaphragm (iv) in Beam 17/3). The literature states that carbonation levels are expected to be low in comparison to inland and sheltered concrete [57, 88, 224, 242], however it is not specified what is classified as “deep” or “shallow” carbonation penetration. Examples of carbonation depths on coastal structures have been previously discussed in Chapter 4. These examples were all found to be less than the levels measured for the current investigation. Indeed, improved correlation was found between the latter results and those from the 1994 Façade report [118], where 19mm was recorded on an internal beam face of an end beam. Based on these observations, the carbonation depths measured for each test beam were considered to be unusually deep. As the depths of carbonation have reached rebar and strand levels, it is expected that this may have contributed to the onset of corrosion on the steel. It is also thought that the longitudinal web cracking had increased the rate of carbonation.

7.4.3 Summary

The outcomes discussed in this section are summarised as follows:

- Elevated chloride levels were determined at all concrete depths for all beams, with most chloride concentrations greater than 0.4% Cl\(^-\) (cement) at steel embedment depth
- Chloride levels did not decrease with increasing depth (as suggested in the literature [21, 131, 170])
- The beam with the least defects, Beam 17/4, had highest chloride concentrations; the opposite was true for Beam 17/3
- It appears that high chloride levels did not correlate with instances of severe corrosion for the current investigation. Indeed, significant corrosion was found for both high and low levels of chlorides, implying a low level of correlation between chloride concentrations and severity of corrosion in reference to limits recommended in the literature [2, 50, 66, 150, 170, 230, 298, 313, 315]
- There appears to be little correlation between carbonation and chloride levels in relation to steel corrosion, especially in regard to chloride binding
7.5 Beam Load Performance in relation to steel condition

7.5.1 Beam performance in relation to serviceability

From estimates made by Sinclair Knight Merz (SKM) in the 1999 Infratech load test report [157], the Sorell Causeway Bridge was found to have sufficient capacity to sustain an occasional T44 design load, that each span was heavily dependent on the load-sharing mechanisms of the diaphragms, and that shear in the diaphragms and beams were not considered to be a critical load scenario. The condition of Span 17 was also not considered critical, with the report stating that Span 17 was operating in a similar manner to that observed for Span 1. The ultimate unfactored flexural capacity of the beams determined by SKM was 544kNm (taking into account prestress and time-dependent losses). The maximum moment achieved by each beam has been discussed in Chapter 5, but is reproduced in Table 7.2.

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>B17/4</th>
<th>B17/3</th>
<th>B118</th>
</tr>
</thead>
<tbody>
<tr>
<td>SKM Ultimate Moment (kNm) [157]</td>
<td>544</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual Moment (kNm)</td>
<td>483</td>
<td>243</td>
<td>333</td>
</tr>
<tr>
<td>% of SKM</td>
<td>89%</td>
<td>45%</td>
<td>61%</td>
</tr>
<tr>
<td>% of Beam 17/4</td>
<td>-</td>
<td>50%</td>
<td>69%</td>
</tr>
</tbody>
</table>

As discussed in Chapter 5, all beams fail to achieve the anticipated SKM target, with the performance of Beam 17/3 representing the most significant departure achieving only 45% of the SKM estimate. More alarmingly was the mode of failure; the brittle nature of the failure of Beam 17/3 and induced crack patterns suggests that this beam failed in shear. Thus whilst the Infratch report [157] stated that shear was not considered critical for the bridge beams in theory, the reality was starkly different for beams undergoing corrosion of both the prestressing strands and shear ligatures. This scenario was highlighted in the 2000 TUNRA report [189], which states that in the event of a sudden, catastrophic failure of a beam with corrosion due to a local load (e.g. a wheel load), impulse or shock loads may be induced on adjacent beams and transverse tendons (up to twice the static load) perhaps leading to the partial or whole collapse of the span. However the report recommended that this scenario was an unlikely failure mode. Based on the load tests conducted, the performance of Beam 17/3, the fact that Beam 17/3 was located under
the wheel path, and that wheel loads from a T44 design load was a maximum of 96kN, it is considered that this scenario presented a very real threat to the safety of the bridge where beams showed evidence of longitudinal cracking (especially since shear strength is strongly reliant on the capacity of the prestressing strands).

In addition to this scenario, the case of several beams overloading is considered. It was noted in Chapter 6 that several strands from Beam 17/3 and 118 had completely corroded whilst in service. This means that these beams (and those of similar condition in the bridge) were more likely to have shed their load onto adjacent beams via the diaphragms. Reviewing strain data for Span 17 from the 1999 Infratech report [157], it appears that a significant proportion of the loads picked up by Beam 17/3 were being transferred to other beams. Taking into consideration that Beam 17/4 did not achieve the SKM target, it is likely that beams adjacent to Beam 17/3 were significantly overloaded, subsequently reducing the factor of safety on this span in relation to the T44 and other legal design loads. If this were the case, then the safety of Span 17 whilst in service may have been severely compromised.

Of course, the transfer of loads relied heavily on the condition of the transverse prestressing strands, of which was a primary concern for DIER. As previously noted in Chapter 6, the condition of the strands retrieved from cores taken through the diaphragms showed (by inspection) that they were on average in reasonably good condition, with the majority of strands centrally located in the tendons and completely surrounded by grout. Transverse strands retrieved from Diaphragms (iv) of both Beams 17/3 and 118 showed more evidence of corrosion however section losses were minor (by inspection) in comparison to those observed for the longitudinal strands. No significant pitting was observed on the transverse strands. Based on these observations, it may be that the condition of the transverse prestressing strands were in better condition than those from longitudinal tendons, perhaps due to the position of the strands in the tendons and the protection offered to the tendons from the size of the diaphragm, and as such may not have posed a serious threat to the serviceability of the structure. However, this does not take into account the grout voids observed in these tendons (see Chapter 4) and previous inspection reports that had identified spalling and corrosion at transverse anchorage ends directly exposed to the elements (see Chapter 2). As a result, future failure of the transverse strands may
have been a reality.

### 7.5.2 Beam performance in relation to corrosion

It is well documented in the literature that corrosion of reinforcement is associated with a reduction in load capacity for structural concrete [76, 318]. If this were not the case, then there would have been no motivation behind the in situ load tests conducted on the Sorell Causeway Bridge whilst still in service. However, what remains the subject of many investigations is the relationship between the load capacity of a reinforced or prestressed concrete beam and the amount of corrosion that has occurred on the steel.

In light of the load-deflection plots in Chapter 5 and the corrosion observations made in Chapter 6, it appears that the ultimate load of Beam 17/3 had been significantly reduced due to the action of corrosion, primarily on the prestressing strands. However, the degree of correlation between section losses and loading characteristics of the beams is yet to be discussed. To compare the ultimate load with the amount of steel corrosion, the minimum average cross-sectional areas of the prestressing steel at the failure point were considered. Also reviewed was the extent of corrosion along the prestressing strands. As conventional reinforcement contributed little to the overall capacity of the beam, discussions shall focus predominantly on the prestressing strand condition.

Table 7.3 provides a statistical overview of the extent and severity of corrosion measured on the prestressing strands for each beam in comparison with the ultimate loads achieved in the load test. Reference is also made to the prestress condition plots starting on page 205 in Chapter 6. Upon the initial visual inspection of the prestressing strands from Beams 17/3 and 118, it was thought that these beams may have registered similar levels of corrosion from the upper and lower tendons in Bay 5. The observation of average cross-sectional area losses at the worst locations in both beams appears to initially support this hypothesis (see Table 7.3). However, upon further investigation and calculation of total section losses for both tendons contained in Bay 5, it is evident that Beam 17/3 has a greater percentage of strands that have losses of 50% or greater in comparison to Beam 118. This is statistically supported in Table 7.3, which shows that Beam 17/3 has 64% and 10% of strands from the lower and upper tendons respectively in Bay 5 have lost...
over 50% of their cross-sectional area in comparison with 37% and 0.5% in the lower and upper tendons respectively from Beam 118.

Table 7.3: Load test results in relation to average steel condition

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>B17/4</th>
<th>B17/3</th>
<th>B118</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Load (kN)</td>
<td>112</td>
<td>57</td>
<td>77</td>
</tr>
<tr>
<td>Ultimate Moment (kNm)</td>
<td>483</td>
<td>243</td>
<td>333</td>
</tr>
<tr>
<td>Average Cross-sectional Area Loss of worst section (Lower Tendon), %</td>
<td>9</td>
<td>80</td>
<td>73</td>
</tr>
<tr>
<td>Average Cross-sectional Area Loss of worst section (Upper Tendon), %</td>
<td>15</td>
<td>48</td>
<td>41</td>
</tr>
<tr>
<td>Percentage of Lower Tendon with Section Losses Greater than 50%</td>
<td>0</td>
<td>64</td>
<td>37</td>
</tr>
<tr>
<td>Percentage of Upper Tendon with Section Losses Greater than 50%</td>
<td>0</td>
<td>10</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The load behaviour of both beams and the subsequently derived load-deflection plots appear to reflect these findings (see Figure 5.3 in Chapter 5). Whilst both beams failed in shear about Diaphragm (iv) at the longitudinal crack site, Beam 17/3 failed catastrophically and suddenly, with strands from both the upper and lower tendons failing in a brittle fashion at approximately the same time. In contrast, Beam 118 took longer to fail with the progressive failure of strands at the crack site. A combination of ductile and brittle failures were observed on strands in the vicinity of the beam failure. Therefore, it appears that the quantity and extent of corrosion observed on the prestressing strands from Beams 17/3 and 118 has influenced the ultimate load achieved and the mode of failure of the beams.

Beam 17/4 presented an interesting case. During load testing, this beam failed in flexure about midspan but at a maximum load less than expected (as previously discussed). Upon inspection of the strands from the failure zone, there was no evidence of corrosion and as such the impact of corrosion on the beam’s capacity could not be assumed automatically. However with the demolition of Bay 5 from Beam 17/4, where the majority of corrosion-related defects were observed, the strands retrieved from this section strands showed a minimum of almost 10% area reduction at the worst corrosion site (as seen in Table 7.3).
Whilst it remains uncertain the exact impact the observed corrosion may have had in relation to the capacity of the beam, it does appear to have contributed to its reduced load performance, and as such this beam would not have been sufficient to support the in-service loads recommended by SKM/Infratech [157].

As a final comment, it is thought that the severe (and in some places complete) section losses of shear ligatures adjacent to longitudinal cracking along Bay 5 may have contributed to the shear failure of Beams 17/3 and 118, and in particular the catastrophic failure of Beam 17/3. Overall, Beam 17/3 had the most number of shear ligatures that had complete section losses, despite the fact that corrosion on ligatures from Beam 118 was more widespread.

### 7.5.3 Summary

The following points are made in summary of this section:

- All beams failed to achieve the anticipated maximum flexural capacity (as determined by SKM/Infratech [157])

- It is likely that Beam 17/3 (and Beam 118) were shedding the majority of their loads to adjacent beams whilst in service due to their poor condition, leading to overloading of these beams

- The transverse strands observed in each test beam (located in the diaphragms) on the whole appeared to be in reasonable condition, however due to the observation of voids and some strand corrosion in Diaphragm (iv) of Beam 17/3, this may cast doubt on the assumption that the in-service performance of the diaphragms was sufficient

- The condition of the longitudinal prestressing strands significantly impacted on the flexural capacity of the beams; the greater the degree of corrosion, the lower the flexural capacity, and the more likely the beam was to fail in shear

- The complete section losses of the shear ligatures within longitudinal web crack zones are thought to have contributed to the “brittle” failures of Beams 17/3 and 118
7.6 The Possibility of Microbiological Influenced Corrosion

While most of the corrosion observations noted in the present investigation can generally be explained by conventional wisdom, there have been some unusual findings that are not. These observations have been summarised below from Chapter 6.

- bright, metallic surfaces either as superficial layer or at the base of deeper pits (surfaces were typically silvery-gold or gun-metal grey)
- wet, black rust (which was sometimes found covering the above-mentioned metallic surfaces)
- cup-like, hemispherical pits, sometimes forming at the base of a larger pit with bright metallic surfaces
- fine, concentric, regularly spaced rings stepping down inside bright, metallic pits
- detection of traces of sulphur elements within and adjacent to bright, metallic pits, including iron sulphide, from XRD/SEM analysis
- mounding of corrosion products
- unusual, aggressive pitting, with the appearance of small channels or tunnels, or patterns similar to a thumbprint
- response of IRB tests implicating the presence of bacteria at the steel surface

A possible explanation of the above observations may be that the corrosion of the steel has been influenced by microbiological activity. This has previously been suggested in Chapter 3, which stated that whilst there have been no specific cases in the literature to prove that microbiological influenced corrosion (MIC) is involved in the corrosion of steel in reinforced concrete, there is no reason why this scenario should be discounted. Thus attention is turned to the aforementioned evidence in light of MIC literature.
There is some evidence to suggest that sulphate reducing bacteria (SRB) and iron related bacteria (IRB) may have been present at the steel surface, or may have even participated in the corrosion process (Chapter 6). For the survival and colonisation of bacteria, they require [172]:

1. water
2. nutrients
3. electron acceptors

Electron acceptors include metals, especially iron. Nutrients include carbon, nitrogen, sulphur and phosphorous. Sulphate reducing bacteria, as the name suggests, reduces sulphates to sulphides (in particular hydrogen sulphide and iron sulphide). For mild steel exposed to marine environments and undergoing MIC, the classical model suggests that the supply of sulphates is from seawater. This may be possible for the beams from the Sorell Causeway Bridge, with seawater occasionally supplied directly to the prestressing strands via the observed longitudinal web cracks. However with the existence of the gypsum-based grout (containing high levels of sulphur, see Chapter 5), this may be a more likely nutrient source.

The unusual pitting morphology observed on some bars may be indicative of SRB activity. For example, the ringed structure of the pitting observed in isolation on ligatures and longitudinal rebars is a known and distinctive characteristic of SRB influenced corrosion. Reference is also made to pitting observed in carbon-steel alloy gas pipelines by Pope as cited by Little and Lee [172]; photographic evidence is presented of pitting associated with microbiological activity, showing hemispherical pits with similarly shaped pits on a smaller scale at the base of the surface. Whilst it is premature to draw conclusions from a single photograph of an unrelated case, it is interesting to note the similarities in pitting morphology between this case and some pits observed on rebars during the course of the current study.

XRD and EDS analysis of such evidence during this project has revealed the presence of Sulphur, chloride-based Green Rusts [GR(I)], base Iron metal, and Iron Sulphide, all of which are known in the literature to be affiliated with MIC, and in particular SRB. There
is also evidence to suggest the evolution of hydrogen gas, possibly by bacterial influence, with observations of brittle corrosion surfaces adjacent to and encompassing pits found across strands and rebars. A notable absence in the case for SRB, however, was that hydrogen sulphide (\(H_2S\)) gas was not detected during the dissection of the beams. It is possible that only small concentrations of gas existed close to pit sites or had diffused into the adjacent concrete previously, but this is unconfirmed. In most cases where bright metal surfaces with an overlaying film of wet, black rust were uncovered, a strong and distinct metallic odour was encountered.

Regarding the irregular channels observed in severe pitting from Figure 6.13, this bears resemblance to tunnelling corrosion. This form of corrosion has been observed on the corrosion of stainless steels by nitric acid, but no specific cases have yet been cited for carbon steel. There is some evidence in the literature (albeit very limited) to implicate the activity of bacteria in tunnelling corrosion [172]. This pitting observation was extremely unusual and no logical explanation can be presented at the present time.

Evidence in the form SEM images showing bacteria present in rust products is well documented [59, 172, 306], however no conclusive evidence was found in SEM images obtained for this thesis. However, Figure 7.9 show a series of images relating to unusual corrosion products located in or adjacent to bright, metallic pits, which may be representative of iron-oxidising bacteria, or at least their direct actions. Figure 7.9a shows a small iron oxide mound that overlay a smooth metallic surface. Figure 7.9b has been reproduced from Chapter 6 to illustrate the filamentous matter wedged amongst other corrosion products found at the edge of a pit. Figure 7.9c shows a series of elongated, branch-like corrosion products that had accumulated around a clean metallic pit with concentric rings. Intertwined amongst these products were small, thin, stalk-like threads (inset A). The corrosion products were rich in chlorine and iron (as determined by EDS). This observation is reportedly a characteristic of the iron-oxidising bacteria *Gallionella*, which has a tendency to concentrate chloride ions [59]. Note that these observations have not been verified and as such should still be regarded with some suspicion.

In concluding this section, it is difficult to say categorically that microbiological activity was involved in the corrosion of both the prestressing and conventional reinforcement in
the current study due to the lack of definitive evidence. If such activity were correct, however, the question still remains as to the influence of the bacteria i.e. are these microorganisms directly involved in the corrosion of the steel, or are they attracted to the steel surface due to the abundant supply of electron acceptors (iron)? However, with unexplained corrosion observations, inconsistent trends in typical corrosion mechanisms (such as chloride concentrations and carbonation levels), and the very high corrosion rates observed for the present investigation, it is recommended that further research be conducted on reinforced and prestressed structures to determine whether such microorganisms are present and their involvement (if any) in the corrosion process.

7.7 Summary

In summarising the key findings in the present investigation, it appears that the traditional methods for interpreting the likely risk of corrosion on reinforced and prestressed concrete were not successful for this instance. Visual inspections failed to detect the corrosion in strands from Beam 17/4. NDT methods may have highlighted zones of likely corrosion when considering the beam holistically, but the results are ambiguous and conflicting. Results also did not comply with the recommended threshold limits and interpretive guides, and showed a low degree of correlation between the results and the physical steel condition. Chloride and carbonation levels reflect similar inconsistencies, with high chloride concentrations not necessarily translating to severe, localised corrosion. All beams did not achieve the anticipated maximum design load, with corrosion significantly impairing the load performance of Beam 17/3. In relation to the pre-demolition investigative data obtained for the Sorell Causeway Bridge by various consultants on behalf of DIER, it appears that these investigations underestimated the poor condition of the bridge. Bearing in mind that this is perhaps the first post-World War II structure to be demolished in Australia, it is recommended that further investigations be conducted on similar existing structures to validate the findings from the current study.
7.7 Summary

(a) Iron oxide growth on clean metal surface

(b) Filamentous matter found at edge of pit

(c) Small stalk-like threads found intertwined products rich in iron and chlorine

Figure 7.9: SEM images showing possible evidence of Microbiologically Influenced Corrosion
Chapter 8

Conclusions & Recommendations

8.1 Conclusions

The Sorell Causeway Bridge was demolished after 45 years of service due to concerns regarding the extent and amount of corrosion on the post-tensioning strands in the precast I-beams. Prior to its decommission, numerous investigations were conducted to establish the condition of the strands and to predict the remaining life of the structure. Despite the detail of these costly investigations, the results were inconclusive. To validate the present findings, further detailed investigations were conducted on three beams salvaged from the bridge after demolition. The aim of the present investigation was to determine the degree of correlation between results from the pre-demolition investigations and the physical condition of the reinforcement and the condition of the concrete in the areas surrounding the reinforcement. Load tests were conducted to establish the extent of capacity loss comparable to reinforcement corrosion.

In summary, the main findings are as follows:

1. There was little visual evidence on the exterior surfaces of the concrete to suggest the severity of the prestressing strand corrosion. Whilst the cracking on Beam 17/3 was significant, with crack widths of up to 20mm, there was no such obvious cracking for both Beams 17/4 and 118 to indicate the severe cross-sectional area losses on prestressing strands. Similar disparity was found for the condition of the shear ligatures and longitudinal reinforcement, which in places, showed complete and localised section loss. Very little rust staining and corrosion product was evident...
from prestressing strands despite significant steel section losses. This raises the question of where these products may have gone, or what they may have transformed to.

2. Both the half-cell potential and resistivity tests were found to be inconclusive in relation to limits usually given in the literature for the prediction of likely corrosion zones and likely corrosion rates. Inconsistencies were also found between the physical condition of the steel and the half-cell potential and resistivity results, even when using the interpretive guides recommended in the literature. For the present test program at least, both test methods appeared to be incapable of identifying zones where pitting was observed to be prevalent. It is recognised that for the resistivity test, errors in the order of a multiple of 0.5 to 2 may have occurred due to the presence of steel, geometric constraints and carbonation effects. However due to a lack of interpretive guides these errors remain unquantified.

3. The concentration of chloride ions was found to be extremely variable. It was also inconsistent in relation to observations of steel corrosion. High levels of chlorides did not necessarily correlate with severe corrosion (especially pitting); and, conversely, corrosion also was found in areas of low or negligible chloride levels. In addition, the literature states that bound chlorides may participate in the corrosion process after being released due to carbonation. However, based on the variability observed in chloride and carbonation results for the present study, the contribution of the calcium chloride additive is unconfirmed.

4. The deep carbonation levels measured for the present investigation were consistent with those reported by Façade Engineering in 1994. Carbonation depths had reached the prestressing strand levels of the lower tendons in some locations for Beams 17/3 and 118. Minimal carbonation was determined across the lower flange; the greatest depths were measured at web/upper flange junctions and towards diaphragms. This indicates that the orientation and geometry of the beams has been influential in the development of the carbonation front. Minimal carbonation was evident in the grout, particularly where tendon voids were present.

5. The load tests showed that none of the beams achieved its design capacity; this included the beam in the best condition (Beam 17/4). Corrosion was found to
significantly impair the ultimate capacity and the ductility of Beams 17/3 and 118, which both failed in a brittle fashion at the longitudinal web crack sites.

6. XRD analyses of corrosion products obtained from both the conventional reinforcement and the prestressing strands showed phases of magnetite ($Fe_3O_4$), goethite ($\alpha - FeOOH$), lepidocrocite ($\gamma - FeOOH$), akaganeite ($\beta - FeOOH$) as well as chloride-based Green Rust (I) ($Fe_{2x+y}Fe_y(OH)_{3x+2y-z}(A^-)$) and Iron (III) Oxide Chloride ($FeOCl$). These products were sometimes found accumulating preferentially around aggregate pieces adjacent to the steel, which is consistent with other findings in the literature.

7. The phenomenon of “chloride weeping” (commonly observed by archaeologists), or beading of ferrous chloride solution, was noted to occur between steel/concrete interfaces on freshly cut concrete surfaces. This observation was most prevalent on Beams 17/3 and 118. The solution was highly acidic, registering a pH of between 3 and 4, which is conducive to steel corrosion (in relation to the classical pitting model).

8. A number of unusual and unexplained observations were made during the course of experimental work. These included:

   - unusual, concave corrosion profiles of prestressing strands from the upper regions of tendons
   - preferential pitting on the underside of prestressing strands from the upper region of tendons
   - bright, metallic surfaces on bars
   - pits with a bright, metallic base
   - wet, black rust overlaying bright metallic steel surfaces containing smaller, concentric pits
   - pits with a series of concentric rings or steps
   - aggressive and deep pitting with the appearance of channels or tunnels
   - positive results for the presence of iron related bacteria
One explanation for these observations is the implication of microbiological activity in the corrosion process, of which many of these observations appear to be similar to typical examples described in the microbiologically influenced corrosion literature.

8.2 Recommendations

Recommendations for further research are as follows:

1. The opportunity to conduct investigations on real-life structures that are more detailed than can be permitted whilst in service is considered a rarity rather than the norm. The majority of data used for providing guidance on the likelihood of corrosion is based on laboratory specimens that may have been subjected to accelerated corrosion - a scenario that does not take into account the service loading or true environmental conditions encountered in real life. Therefore it is recommended that further opportunities be created to assess “real-life” structures in preference to laboratory specimens.

2. It may also be prudent to establish a national database which collates physical data collected from field investigations. With this information, it may be possible to clarify the degree of correlation between pre-demolition investigations and the physical condition of the reinforcing steel. In addition, it is also recommended that existing field data be reviewed to confirm whether similar data inconsistencies have been observed elsewhere. This is especially recommended for structures with chloride contamination which currently is widely used as a crucial indicator for assessing the risk of corrosion.

3. Whilst half-cell potential and concrete resistivity surveys remain popular field investigation techniques, it is recommended that investigations be conducted into the degree of correlation between physical condition of the steel and the Linear Polarisation Resistance technique, a method that is being increasingly used for corrosion risk assessments.

4. Further development into the interpretation and the correction of errors pertaining to the resistivity method is required.
5. It is recommended that the phenomenon of “chloride weeping” and the evolution of ferrous chloride be further investigated to see if the occurrences observed in the present research is unusual or a normal step in the corrosion process that is often missed in field investigations.

6. Due to the unusual observations made in the present investigation, further studies are suggested in the field of microbiologically influenced corrosion to determine whether bacteria are present during the corrosion process in reinforced and prestressed concrete, and whether these microorganisms play an active role in the corrosion process.
Appendices
Appendix A

Beam Selection & Nomenclature

A.1 Beam Selection

As part of the tender documents, the successful contractor to construct the new bridge was required to set aside eight beams from the demolition process for future research. Beams from Span 17 were salvaged, which included Beam #12 which had the most significant longitudinal web cracking observed for the entire bridge. These beams were stored in a DIER-owned quarry not far from the bridge site. The remainder of the beams retrieved from the demolition process became the property of the demolition contractor, Hazel Brothers, and were stored at the contractor’s quarry site.

From the outset of the project, the decision was made to test three to four beams of varying condition, ranging from a good condition to severe. Two beams from Span 17 were selected on this basis; one being the best condition and one being the worst. These beams were situated side-by-side whilst in service. A third beam of intermediate condition was selected from the beams stored at the Hazel Brothers quarry. Once chosen, the beams were relocated from Tasmania to the Structural Engineering Laboratory at the University of Newcastle for testing.

Table A.1 shows a summary of the beams, detailing their location in the bridge, beam number and casting date, and a brief overview of their visual condition. Each beam was
assigned an individual beam number for the duration of the project. The location of the intermediate beam, Beam 118, was difficult to identify, as no span number was recorded for the beam during demolition. However, based on casting numbers and the dates and span locations of other beams retrieved from the bridge demolition, Beam 118 can be placed between spans 6 and 8. Reviewing DIER inspection information and the defect locations noted on Figure 2.13, it is hypothesised that the beam is either Beam #13 from Span 6, Beam #8 from Span 7, or Beams #7 or 11 from Span 8. The location of the beams are shown in Figure A.1. Beam nomenclature is discussed in Section A.2.

Table A.1: Test specimen identification and description

<table>
<thead>
<tr>
<th>Beam Number</th>
<th>Beam Cast #</th>
<th>Casting Date (1956)</th>
<th>Location</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>B17/4</td>
<td>254</td>
<td>17 Aug</td>
<td>Span 17, Beam #11 (or #4)</td>
<td>Good; minor spalls over ligatures</td>
</tr>
<tr>
<td>B17/3</td>
<td>256</td>
<td>18 Aug</td>
<td>Span 17, Beam #12 (or #3)</td>
<td>Poor; severe longitudinal web crack, matching crack along soffit</td>
</tr>
<tr>
<td>B118</td>
<td>118</td>
<td>18 May</td>
<td>Unknown; Between Spans 6 and 8</td>
<td>Reasonable; smaller longitudinal crack; spall at end bay</td>
</tr>
</tbody>
</table>

**A.2 Nomenclature & Lot Numbering**

For consistency in results, lot numbering and naming convention systems were established. This section provides details on the development of the nomenclature which is referred to elsewhere in this thesis. Regarding the beam name, the original beam location was incorporated into the nomenclature; for example, Beam 17/3 is Beam #3 (or #12 in DIER records, and as shown in Figure A.1) from Span 17. Spans are numbered sequentially from the western abutment, and beams are numbered from the southern or seaward side of the bridge. For Beam 118, the beam casting number was used for its nomenclature (as the exact location remains unknown).
Figure A.1: Location of beam specimens in overall schematic of the Sorell Causeway Bridge
A.2 Nomenclature & Lot Numbering

An arbitrary numbering system was established for the definition of the beam geometry, shown in Figure A.2. Each beam face was allocated a “face number”; each end of the beam was either “Face A” or “Face B”; the upper and lower edges of the beam correspond to “Face E” or “Face F”, and each side face “Face C” or “Face D”. The latter nomenclature is most frequently used in experimental result descriptions. Each beam has been divided into a series of “bays”, which reflect the portions of the beams between each diaphragm. Bays are numbered 1 to 6. Intersecting diaphragms were given the numbers (i) to (v). In practice, beams were aligned in the same manner as they existed whilst in service; thus Face A signified the western-most end of the beam and the defects observed in each beam were similarly aligned in Bay 5.

The nomenclature for each beam cross-section is shown in Figure A.3. The cross-section was divided into components of upper and lower flanges and the web (as seen in Figure A.4a); note that the webs are represented by Faces C and D. Steel types were grouped in accordance with their function; the longitudinal reinforcement and shear ligatures have been termed “conventional reinforcement” or “rebars”, and post-tensioning steel “prestressing strands” or “prestressing steel”.

Each section of steel was also labelled in relation to its position within the beam cross-section, as shown by Figure A.4b. For example, conventional reinforcement labels reflect the position of either Face C or D and prestressing strand position is indicated by upper or lower tendons. Prestressing strands from each tendon were also numbered individually in order to keep track of its location; this is shown in Figure A.5. Note that the predominant reference for prestressing strands used in the present investigation relates to those strands from the longitudinal tendons. For prestressing strands from the transverse tendons (i.e. those located through the diaphragms) these are referred to as one section, either from the upper or lower transverse tendon (depending on the diaphragm).
A.2 Nomenclature & Lot Numbering

Figure A.2: Allocation of lot numbering for beam geometry - Face C plan view

(a) Beam 17/4

(b) Beam 17/3

(c) Beam 118

Figure A.3: Nomenclature for each beam - Face D plan view
A.2 Nomenclature & Lot Numbering

(a) Nomenclature for the Beam Cross-section

(b) Nomenclature for the Steel

Figure A.4: Beam Cross-section Nomenclature - from Face A

Figure A.5: Numbering of prestressing strands within tendons
Appendix B

Preliminary Information & Methodologies

B.1 Introduction

This appendix contains details relating to background information and methodologies of the inspection techniques employed for the present investigation. Most of the information presented relates to project specific information that is not covered by the relevant standards and documentation for each method.

B.2 Visual Inspections

B.2.1 Relevant Standards and Guides

There is no specific standard providing guidelines for visual inspections, although many infrastructure managers have devised their own requirements as part of an asset management plan. For the present study, several resources were utilised prior to conducting the visual inspections, including “Guide for Evaluation of Concrete Structure Prior to Rehabilitation” [4], “Monitoring of steel corrosion in concrete” [50] and “Guide to Concrete Repair and Protection” [150]. Project-specific procedures are discussed in Section B.2.2.
B.2 Visual Inspections

B.2.2 Methodology

The purpose of the visual inspection for the present study was to:

- confirm the geometric dimensions of the beams as required by the design specifications
- accurately locate and describe visible defects (including incidences of rust staining)
- conduct mapping of significant cracks in all beams

Prior to the commencement of the visual inspection on each beam, the collation of projectspecific documentation was required in order to review past defects. This included the review of all design and construction documentation (including as-built drawings) and previous inspection records (including photographic evidence of defects). The majority of this information has previously been reviewed in Chapter 2. Inspection tools for this investigation included:

- inspection templates (for logging defects)
- SLR digital camera
- tape measure and ruler
- profile gauge (for measuring depth of spalls)
- a crack gauge

The visual inspection commenced with an overall dimensional check of the beams. In summary the beams were found to comply with the design specifications, with errors no greater than ± 5mm. A detailed record of defects was then conducted for every beam surface. Surface defects such as surface laitance, non-compacted or “boney” concrete, significant blow holes or poor concrete quality, efflorescence, spalling, cracking and rust staining were recorded, noting the location, size, and extent of the defects. Where spalling and corroding reinforcement was observed, the profile gauge was used to measure the depth of cover to the bars.

Crack mapping was conducted in more detail than the other defects. Mapping included locating the extent and the detail of the cracks relative to the web or base flange surface.
on which it occurred. Crack widths were measured at 50mm increments using the crack gauge.

Thorough photographic records were obtained for every surface of each beam. This included overviews of each bay and detailed photographs of defects. This not only allowed the recording of defects prior to load testing, but enabled the stitching of photographs for later use in Chapters 4, 5 and 6. Photographic and video records were also obtained of each beam before and after load testing to observe the behaviour of the beams and defects under load. These records are available in Appendix C.

B.3 Covermeter Survey

B.3.1 Relevant Standards and Guides

There are no specific Australian or ASTM standards relating to the use of covermeters. However the British Standards Institute has released BS1881:204 [73] which details the use and limitations of taking cover readings. This methodology has largely been adopted for the present study. The Proceq PROFOMETER4 manual was also utilized for guidance [229]. The cover specifications have been outlined by Figure 4.12 in Chapter 4.

B.3.2 Equipment

A wide variety of covermeters have been developed since its inception in the 1950’s, improving with accuracy and functionality over time. For the present investigation, the Proceq PROFOMETER4 (Model S) was used, as shown in Figure B.1. It comprises a digital meter (which also serves as a datalogger) connected to a flat search probe containing the electromagnet.

B.3.3 Method & Interpretation

Prior to conducting the cover survey, the covermeter was calibrated using standard procedures employing a calibration block made by the The University of Newcastle laboratory staff specifically for this purpose. It included the full range of bar diameters and covers specified for the bridge beams. Additional on-site calibrations were conducted on beams
B.3 Covermeter Survey

The majority of covermeter surveys on each beam were conducted prior to load testing; web and upper and lower flange surfaces were surveyed with the general exclusion of diaphragm surfaces. Bar diameters were pre-set into the covermeter prior to taking respective readings. Surveys predominantly focused on the location and depth measurement of the shear ligatures, measuring the cover in the webs at 100mm vertical spacings. Horizontal readings were governed by the physical spacings of the ligatures. Each point measured was given a number and its position along the web recorded manually.

A similar process was adopted for the base flange soffit surveys. Two surveys were conducted over the same area in order to detect the covers for both longitudinal reinforcement and shear ligatures. Attempts were also made to measure the cover of the prestressing strands. The location of the tendons were determined, however due to the large size of the tendons (approximately 50mm) and the strand congestion, accurate cover readings were not obtained. Reinforcement placement from several freshly-cut beam surfaces for each beam were also measured to determine the degree of correlation between the covermeter survey and the physical geometry of the reinforcement.

Upon the completion of the surveys, all data from the covermeter was downloaded into
B.4 Half-Cell Potential Survey

B.4.1 Relevant Standards and Guides

For reinforced concrete structures with uncoated reinforcing bars, the most commonly used standard for the half-cell potential test is the “Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete” (ASTM 876) [43]. There are no corresponding Australian or British standards. The RILEM Technical Committee TC-154 has also issued a recommendation which include equipment recommendations, methodology and assistance in result interpretation [244]. Further advice regarding half-cell potential data interpretation was obtained from Chess & Gronvold [93] and Vassie [302].

B.4.2 Equipment

The half-cell potential test requires relatively simple equipment that is cheap and readily available. It requires a high impedance 10mV voltmeter, connecting the positive terminal to the reinforcement (to ensure a negative potential reading) and a standard reference electrode. There are a number of standard reference electrodes available for use, however the electrodes most commonly used for field investigations are:

- Calomel (SCE) (a mercury rod immersed in a mercuric chloride solution)
- Copper-Copper Sulphate (CSE)
- Silver-Silver Chloride (SSCE)

The Copper-Copper Sulphate reference electrode was preferentially used for the majority of readings (to correspond to DIER results), however a small number of results were obtained using the Silver-Silver Chloride reference electrode. The tip of the electrode is covered in a detergent-soaked sponge which is applied to the concrete surface. The complete equipment configuration is shown in Figure B.2.
B.4 Half-Cell Potential Survey

Figure B.2: Half-cell potential equipment configuration

B.4.3 Methodology

The web faces and soffits of all beams were surveyed for half-cell potentials. The majority of potentials were obtained prior to load testing with the exception of Bay 5 of Beams 17/3 and 118. Survey grid spacings were fixed at 100mm, which is smaller than the recommended 1200mm spacings by ASTM C876 [43] and 500mm by RILEM [244]. It is usually thought that a small grid spacing of this size will identify zones of micro-cell corrosion [54, 75].

Steel connection points were generally restricted to one per beam where possible, with the exception of Beam 17/4 which required two connection points (due to accessibility issues). Exposed reinforcement was used where possible to minimise the concrete damage prior to load testing, however where this was not possible small areas of concrete were removed to reveal the embedded steel. Longitudinal reinforcement from the upper flange was predominantly used as a connection point and prior to taking any readings, electrical connectivity tests were conducted. All steel surfaces were cleaned free of rust deposits at connection points to ensure a good electrical connection.
Prior to carrying out the surveys, the concrete surface was pre-wetted to minimise the resistive effects of the concrete and assist in stabilising the potentials. Pre-wetting was not always necessary as the sponge covering the tip of the electrode was usually saturated with a diluted solution of detergent [43, 244]. To determine whether pre-wetting was required, the electrode was pressed onto the surface of the dry concrete. If the potential readings were stable, no pre-wetting was required. However, the majority of potentials recorded did fluctuate which necessitated pre-wetting. Surfaces were pre-wetted using a spray bottle containing detergent water.

The potential readings were taken by pressing the reference electrode onto the pre-marked grid spacings. Potentials were recorded manually once the readings had stabilised. In some cases, potential drifts were observed; the majority were less than 20mV which can be ignored according to RILEM [244], however, where results continued to fluctuate, surfaces were re-wetted and the reading was taken again. Steel connection points, adjacent defects, temperature, and relative humidity were also noted on the data sheets.

Additional half-cell potentials were obtained for Bay 5 of Beams 17/3 and 118, carried out after load testing. This involved a variation in the steel connection points to observe the variation in half-cell potentials [217]. This included strands from the upper and lower tendons and varying longitudinal bars.

B.4.4 Data Representation & Interpretation

**Representation**

There are a number of ways in which the half-cell potential data can be represented, depending on what is required. ASTM C876 [43] and in particular RILEM [244] provide good reviews on each method, which are outlined briefly below.

Frequency graphs/histographs present data in a statistical manner with respect to the number of readings divided into potential groups. This enables identification of the most frequently occurring potential for the entire beam, but it does not provide information on the location of the potentials. Potential contour maps (or equipotential plots) represent
B.5 Concrete Resistivity Survey

data in a graphical manner. These can be in the form of black and white equipotential contour lines, coloured surfaces or as a three-dimensional plot. These methods have been used herein for data representation.

Interpretation

Vassie [302] recommends that interpreting the absolute value of the potential readings in relation to the ASTM C876 limits [43] should not be considered in isolation or taken as a direct indication of the likelihood of corrosion. Values should be reviewed in light of the recommendations outlined by Chess & Gronvold [93], RILEM [244] and Vassie [302].

These recommendations suggest that the analysis of potential gradients and local maxima and minima will yield information pertaining to corrosion activity. For example, steep or closely-spaced gradients and minima are likely to reveal areas where corrosion is occurring. Where carbonation of the concrete has occurred, corrosion may be difficult to identify (refer to Table 4.4 in Chapter 4), however for the purposes of this research it was anticipated that corrosion zones may be easily identifiable due to the small grid size selected. Careful assessment of results was required due to high resistive surface layers, which were encountered on web surfaces during chloride drilling (see Chapter 5).

Where positive values were encountered, this may have been the result of a variety of factors, including the concrete is very dry, the concrete is free of chlorides, stray currents are present or that the electrical connection to the steel is incomplete [75, 116, 302]. Lauer recommends that such readings not be included as part of the assessment [169].

B.5 Concrete Resistivity Survey

B.5.1 Relevant Standards and Guides

No Australian, ASTM, or British Standards were found relating to the use of the resistivity test on reinforced concrete structures. The most relevant standard found was ASTM G56 [46], however this relates to the field measurement of soil resistivities. Additional guidance was sought from several other publications, which include a RILEM recommen-
B.5 Concrete Resistivity Survey

dation [243] and various papers published by Millard and colleagues [140, 141, 196–199, 201].

B.5.2 Equipment

Three different meters were used to obtain concrete resistivity results for the current project, all of which centre around the use of the four-point Wenner probe. The meters used were the Proceq RESI resistivity meter (Figure B.15a), Tinker & Rasor SR-2 Soil resistivity meter (Figure B.15b), and the MEGGER® DET5/4D Earth Tester soil resistivity meter (Figure B.3c). The RESI meter provides an electronic datalogger that automatically converts resistance measurements to concrete resistivities. The Wenner probe for this meter comprises hollow copper electrodes spaced at 50mm with inserted sponges for pre-wetting (see Section B.8.3). Information obtained with the SR-2 and MEGGER® meters were recorded manually and required results to be converted to resistivities through the use of Equation 4.1 from Chapter 4. The probe used in conjunction with these meters comprised of spring-loaded brass electrodes spaced at 50mm. Sponges were placed over the tips of the electrodes to reduce the effects of resistive surface layers and improve electrical contact with the concrete surface. Unfortunately, results obtained with the SR-2 meter were discarded due to serious calibration errors and instrument malfunction.

B.5.3 Methodology

Resistivity readings were obtained after each beam was load tested and cut into manageable segments. Vertical surfaces of webs and flanges from all bays were tested with the exception of some sections of Bay 5 for Beams 17/3 and 118. The soffit of the base flange was not surveyed due to the influence of closely spaced reinforcement and boundary effects. A grid of approximately 200 x 200mm was marked up on the webs of each bay, which was predetermined from the covermeter survey. The grid locations were subsequently governed by the shear ligature spacings (approximately 200mm), and vertical grid spacings varied due to the parabolic trajectory of the tendons. Figure B.4 shows some typical examples of test locations.
Readings were obtained by pressing the sponge-tipped electrodes onto the surface of the concrete. The temperature, relative humidity, current weather conditions, and past significant weather events were recorded for each test. Surface conditions and defects were also noted. Care was taken to place the resistivity probe in locations that avoided both the tendons and shear ligatures in an effort to minimise the effects of steel (see Chapter 4). The probe was placed in various orientations where feasible (horizontal, vertical and diagonal) in order to verify the result. Figure B.5 shows examples of probe placements for Bay 5 which conform to the placement of the steel. Resistivity results from a diagonally placed probe were found to be least influenced by steel [66, 243]. In some instances, some
B.5 Concrete Resistivity Survey

Errors were unavoidable due to the irregular placement of reinforcement cages, reduced cover (see Chapter 5), and the fixity of the probe electrode spacing.

Figure B.4: Typical examples of test locations for resistivity testing

Stability was required prior to recording the results, where fluctuations of no more than 1 kΩ.cm (or a resistance value of approximately 60 kΩ for 50mm electrode spacing) are permissible [123]. Readings were re-taken where unstable or erroneous results were obtained.

Figure B.5: Probe placement for resistivity testing

In addition to the above method, superficial holes were drilled into the concrete surface at pre-determined locations where brass electrodes were inserted and the resistivity recorded using the MEGGER® meter. Figure B.6 shows the test setup. Results from both dry and pre-wetted surface conditions were recorded, in order to observe correlations between these two methods and the previously described method. However, the drilled holes were
located horizontally and thus increasing the likelihood of errors in relation to the close proximity of the shear ligatures.

Figure B.6: Resistivity experimental setup for pre-drilled holes using the MEGGER® meter

B.5.4 Representation and Interpretation

*Representation*

As there is no specific standard regarding resistivity measurements of concrete, there are a number of views in the literature on how best to represent resistivity data. The majority of the literature (especially in case studies) report resistivity values as an absolute or average value, with little or no information regarding the test location or environmental conditions at the time of the test [19, 80, 121, 129, 300]. Vassie [301] presents resistivity data statistically using histograms.

The RILEM recommendation [243] suggests that results are best represented using a map or marked up on location sketches. Further to this representation, several references present information as part of equiresistance contour plots [124, 208, 260]. Others present
numerical data in their set grid points \[70, 176\]. All of these methods have been employed for the analysis of resistivity results obtained for experimental results.

**Interpretation**

Interpretation of these results is less complicated than that of the half-cell potentials, however there is little in the literature to assist in this task. The factors listed in Section 4.5.2 in Chapter 4 present some guidance towards results analysis. Cover meter surveys prior to the test theoretically eliminate errors associated with reinforcement, however this error was not always avoidable. Results are reviewed in light of publications by Millard and colleagues which provide guidance in quantifying resistivity errors \[140, 141, 196–199, 201\].

### B.6 Chloride Profiles

#### B.6.1 Relevant Standards and Guides

For the collection of both core and dust samples, several standards were reviewed which included AS1012.14 \[32\], ASTM C42 \[41\] and HB-84 \[150\]. For sample preparation for chloride concentration determination, AS1012.20 \[33\] was predominantly used, with additional reference to BS1881:124 \[72\] and ASTM 1152 \[40\]. The Standards Australia publication HB-84 \[150\] also provided a good information relating to the on-site collection of concrete samples during field investigations.

#### B.6.2 Equipment

Both core and dust samples were retrieved for analysis. Cores were retrieved using the water-lubricated Hilti Coring machine, using a 25mm diameter, 500mm long drill bit. Core samples were cut into pre-set lengths using a diamond-tipped circular saw. Samples requiring crushing prior to chloride analysis (in accordance with AS1012.20 \[33\]) were prepared using an electronic mortar and pestle grinder, as shown in Figure B.7. Dust samples were obtained by using a diamond-tipped drill bit attached to a hammer drill as shown in Figure B.8. The dust sample was collected using a funnel and a sterile container. Equipment used for the preparation and analysis of samples was similar to that described in AS1012.20 \[33\].
B.6 Chloride Profiles

A variety of test sites across Bay 5 for each beam were selected for chloride profiling. The majority of cross-sectional samples were obtained through the web adjacent to the prestressing tendon. A minimum of four sites per beam were chosen for chloride profiling through the web cross-section. A small number of samples were retrieved from the soffit of the base flange from each beam to determine the cross-sectional profile to the longitudinal reinforcement. All cross-section samples were obtained either by small cores or by dust using the methods described in Section B.6.2. Core samples were sliced into 10mm increments which were subsequently crushed to a particle diameter specified in AS1012.20 [33]. Dust sub-samples were retrieved at 10-15mm increments (using pre-set depths fixed by the drill) until the outer perimeter of the tendon was reached.

In addition to the cross-sectional profiles, cores were taken longitudinally through the web following the prestressing tendons for observation of chloride concentrations variations. Five longitudinal cores were obtained for Bay 5 of Beam 17/4; only one core
was retrieved for both Beams 17/3 and 118. Longitudinal cores were sliced into 50mm increments (where required) and ground to a fine dust similar to that observed for the cross-sectional cores. Several grout samples were also selected at random for chloride analysis. These samples were prepared in a similar manner to the longitudinal cores. The locations of all samples for each beam are shown in Figures B.10, B.11, and B.12.

All samples were then prepared in accordance with AS1012.20 [33]. Figure B.9 shows the progress of the samples after acid boiling. The determination of the chloride concentration of each sample was conducted either by titration (as per AS1012.20 [33] or BS1881:124 [72]) or through the use of an Ion Chromatographer which identified the concentration of a number of anionic species from the sample. The results from both methods were found to be in agreement after conducting calibration runs. Chloride concentrations were then calculated for each sample according to its percentage weight in relation to the concrete and cement content. The cement content of the concrete was predetermined from previous investigation reports [110, 118]. The adopted method of representation for the present investigation is % Chloride by weight of cement (%Cl$^-_{\text{cement}}$). Chloride profile plots were then prepared, showing the variation of chloride concentrations with depth or position.
Figure B.10: Chloride sample locations for Beam 17/A
Figure B.11: Chloride sample locations for Beam 17/3
B.7 Carbonation Measurement

B.7.1 Relevant Standards and Guides

There are a limited number of standards published for the determination of carbonation in relation to reinforced concrete. The most commonly used guide is the RILEM CPC-18 recommendation [245]. This recommendation, however, applies to prefabricated specimens and not insitu concrete. The Standards Australia publication HB-84 [150] and the Austroads publication AP-T06 [50] provide some guidance on the method of sampling insitu. Broomfield [66] and Bungey and Millard [77] provide some additional guidance.

B.7.2 Equipment and Methodology

The method for determining carbonation depths is simple, requiring the use of a spray bottle containing a clear solution of 1% phenolphthalein indicator in 70% ethyl alcohol. The solution was sprayed onto freshly cut concrete surfaces, which were cores and beam cross-sections. If the concrete surface has a pH greater than 9.2, then the indicator will colour the surface deep pink. Where concrete has a pH less than this value (i.e. the concrete is carbonated), then the colour of the surface will not change. Photographs of the concrete surface were taken before and after the application of the phenolphthalein indicator solution. Carbonation depths were measured using a ruler approximately 24 hours after the first application.

There is some debate relating to the accuracy of the phenolphthalein indicator, as the carbonation process is considered to commence at a pH of 11 which will not be identified by the indicator. Some literature state that the carbonation front may be almost twice the depth of that determined by the phenolphthalein [66, 90, 160, 174]. However, as the corrosion of steel in concrete is traditionally thought to initiate at a pH less than 9 the phenolphthalein indicator is still considered to be a useful tool for field investigations. To confirm the pH of the concrete adjacent to steel, random pH spot checks were conducted using pH indicator sticks. An example is shown in Figure B.13. Readings were taken by pre-wetting the surface of the concrete with water and pressing the pH tab onto the saturated surface. The tab was then removed and the colour and pH reading was
B.8 Load Testing

B.8.1 Relevant Standards and Guides

Whilst AS5100.7 [38] provides some guidance on the methodology and interpretation of load testing, this applies to testing conducted on structures in-service. There are no specific publications describing the load test process, therefore the methodology contained in ASTM C78-02 [42] was adopted. AS5100.2 [37] was also reviewed for recommendations pertaining to design loads and insitu testing in the lead up to testing. For additional information, Bungey and Millard [76] provide a useful guide relating to monitoring and techniques for ultimate load testing.

B.8.2 Equipment

The equipment setup is shown in Figure B.14. The beams were simply supported on two steel support mounts that were especially fabricated to simulate the original bearing
B.8 Load Testing

plates, with 10mm rubber pad squares inserted between the two surfaces. Two 250kN hydraulic jacks, suspended from a steel support frame, applied the load at third points along the beam; this configuration was identical for each beam. The load points were aligned as close as possible with the beam vertical centreline to attempt to eliminate eccentric loading and to avoid twisting action. Additional lateral restraint was afforded by the support frames to assist in alignment.

![Figure B.14: Load test equipment setup](image)

To monitor the development of strains throughout the loading process, twelve 10mm potentiometers were distributed either side of Diaphragm (iii) on both faces, as shown in Figure B.14. Figure B.15 show the numbering and layout of the potentiometers for Faces C and D. The meters were placed down the face of the web and along the bottom edge of the base flange in order to observe strains across the beam cross-section. Midspan deflections for each face were monitored using two 100mm travel displacement potentiometers placed under Diaphragm (iii).

![Figure B.15: Strain gauge layout at midspan](image)
B.8.3 Methodology

Load tests for each beam were conducted after detailed visual, crack-mapping, photographic, half-cell and covermeter surveys had been carried out. The loading process was controlled remotely via computer console which also logged the load, strain and deflection data. Loads were applied incrementally via load control as dictated by a laboratory technician. Increments were initially stepped at 5kN loads in order to eliminate irregularities, and then proceeded at steps of approximately 10kN, adjusting the increments as the beams approached its ultimate load.

During the process, loading was halted temporarily at various loading stages to allow detailed photographs to be taken of existing and developing crack zones. Stereo photographs were also taken throughout this process at specific sections. This involved the placement of two SLR cameras on tripods at two separate locations focused on the same point. The focal point for Beam 17/4 was Diaphragm (iii) (midspan), and the longitudinal cracks along Bay 5 for Beams 17/3 and 118. A video camera at the end of each beam was also set up to document the loading process.
Appendix C

Experimental Results
REFERENCES


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