Deterioration of Prestressed Concrete Bridge Beams

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Abbreviations and acronyms

FIB: Federation Internationale du Beton
MoW: Ministry of Works
MWD: Ministry of Works and Development
VPV: Volume of Permeable Voids
W/C: Water to cement ratio
XRF: X-ray Fluorescence

Contents

Executive summary

The Hamanatua Stream Bridge was built in 1966 to a standard Ministry of Works and Development design used in approximately 117 State Highway bridges. A routine inspection in 2004 revealed that the prestressing steel on at least one of its beams was corroding. Although the strand has not yet broken, the corrosion has cracked and spalled the cover concrete.

A feature of this beam design is that the prestressing strand is not fully confined by stirrups, which means that corrosion of the stirrups does not provide an early warning of imminent corrosion of the prestressing steel. Because consequences of corrosion in prestressed components can be severe, this investigation was initiated to identify the current and future risks associated with prestressing steel failure in bridges, particularly prestressed concrete beams built in the 1960s and early 1970s. The findings will facilitate informed, cost-effective decisions regarding the future management of such bridges.

The research was carried out in 2005–2006. It aimed to:

- identify the factors that contributed to the deterioration on the Hamanatua Stream Bridge;
- assess the current condition and future risk of corrosion of both prestressing and conventional reinforcement on other bridges of similar age and design in a range of exposure environments in New Zealand to ascertain whether this type of deterioration is widespread;
- assess the variability in materials and workmanship for this type of bridge beam; and
- develop recommendations for the future management of these structures to assist New Zealand bridge owners and managers to optimise the economic life of the bridge stock and the remaining life of individual structures.

An implicit aim was to find out whether the corrosion risk has been reduced by current prestressed concrete beam design standards.

The research involved site assessments of the condition of the Hamanatua Stream Bridge and of 29 bridges of similar design, representing a range of ages and exposure conditions, to ascertain the cause and extent of prestressing corrosion. Measured depths of concrete cover and chloride ion contamination on the beams were used to predict the onset of corrosion and hence whether the bridges would achieve a 100-year service life without intervention to keep them in a serviceable condition.

The quality of the concrete materials and workmanship in the Hamanatua Stream Bridge were found to be generally good. Specified cover depths were one inch (25 mm), much less than current requirements, but the cover to some of the steel was less than 25 mm. The bridge is in the B2 'coastal frontage' exposure zone defined by NZS 3101: 2006 and within 200 m of an open surf beach. The corrosion was caused by the ingress of chloride ions from sea spray, resulting in chloride ion concentrations at the steel surface exceeding the threshold value at which corrosion is initiated. The influence of prestressing steel composition and different corrosion mechanisms on the observed deterioration could not be determined.

Despite the risk of chloride-induced corrosion, corrosion of prestressing steel was not observed on any other bridges of this design, although corrosion of conventional reinforcement was relatively common. In some cases, this may be because the small volume of corrosion products has not generated sufficient stress to damage the cover concrete.

Nevertheless, site and laboratory testing showed that other bridges of the same design in the B2 exposure zone are likely to be affected by the same deterioration mechanism because the chloride contamination, quality of concrete and the depth of cover were similar to those on the Hamanatua Stream Bridge. Corrosion is unlikely in bridges of the same design in the B1 ('coastal perimeter') and A2 ('inland') exposure zones because they are not exposed to external sources of chloride ion contamination.

Analysis of concrete samples from these bridges also showed that the concrete in some of them contains calcium chloride accelerating admixture, which increases the likelihood of corrosion irrespective of exposure conditions.

On bridges where corrosion damage is not yet evident, the approximate time to future corrosion can be predicted by a simple model that uses chloride ion diffusion rates to predict the onset of corrosion (often referred to as 'time to corrosion initiation'). The bond between prestressing steel and concrete may be lost shortly after corrosion begins, so the time to corrosion initiation is a reasonable approximation of time to corrosion damage. The model predicted that bridges of the same design as the Hamanatua Stream Bridge and in the B2 exposure zone are unlikely to achieve a 100-year service life without some corrosion damage, but that corrosion is unlikely within this period on bridges of the same design in the B1 and A2 exposure zones. This broadly correlated with observations. Refinements to the sampling and modelling procedures may allow more precise predictions.

Bridge beams designed to current specifications (NZS 3101: 2006 and Transit New Zealand's Bridge Manual) have much greater cover depths. The corrosion initiation model indicated that beams made with similar concrete quality as the Hamanatua Stream Bridge but with cover depths in accordance with current specifications will probably achieve a 100-year service life without corrosion damage. Beams with similar concrete properties and cover depths to the Hamanatua Stream Bridge do not comply with current specifications and are just as likely to be affected by corrosion.

For a given corrosion rate, corrosion of the prestressing strand reduces the structural performance of a beam faster than corrosion of conventional reinforcing because a greater proportion of the steel cross-section is lost. The Hamanatua Stream Bridge beam on which corrosion was observed may have lost up to 10% of its live load capacity, reducing the overall live load superstructure capacity by up to 5%. The corrosion mechanisms and eventual strand failure mode on the Hamanatua Stream Bridge could not be determined because the strand could not be sampled safely for investigation. Therefore the possible influences of hydrogen embrittlement, stress corrosion cracking, fretting corrosion and fatigue corrosion on the eventual strand failure mode on the Hamanatua Stream Bridge remain unknown.

The risk associated with prestressing steel corrosion in these bridges is greater than the risk of reinforcement corrosion in bridges of similar age. Some intervention will be necessary to ensure that bridges of this particular design in the B2 exposure zone remain serviceable for a 100-year service life. This may involve either preventive maintenance or repair to the concrete once the steel has started to corrode.

To enable Transit New Zealand and LTNZ to cost-effectively and proactively manage the risk in pre-1973 prestressed concrete bridges and in more recent designs, further work is recommended to identify the bridges at risk and to identify appropriate methods of managing prestressing corrosion in them. Further research topics were identified to refine methods of predicting corrosion initiation and to optimise mitigation strategies.

Abstract

A routine inspection revealed significant corrosion of the prestressing strand on a concrete road bridge built in 1966 to a standard design used in about 117 State Highway bridges in New Zealand. To identify the cause of the deterioration and how many bridges of this design might be affected, the conditions of 29 similar bridges on New Zealand State Highways were evaluated by site investigation. The research, carried out in 2005–2006, found that although the concrete quality in the bridge beams was generally good, the combination of cover depths less than 25 mm and exposure to salt spray had increased the likelihood of corrosion in bridges of this design in the B2 (coastal frontage) exposure zone. Bridges in the B1 (coastal perimeter) and A2 (inland) zones are less likely to be affected, although the concrete in some of the beams contained chlorides added during construction. The risk associated with prestressing corrosion in this beam design is higher than in current designs because the prestressing strand is poorly confined and the cover depth is low. Bridges of this design in the B2 zone will probably need some form of intervention to remain serviceable for a 100-year service life.

1. Introduction

1.1 Background

The Hamanatua Stream Bridge was built in 1966 on State Highway 35 near Gisborne on the East Coast of the North Island in New Zealand. It is a prestressed (pretensioned) structure[1]. In 2004, a routine detailed inspection of the Hamanatua Stream Bridge identified significant deterioration of one of the prestressed concrete beams whereby corrosion of the outer prestressing strand on the bottom flange of the beam had spalled the cover concrete along much of the length of the strand. Further investigation by the regional bridge consultant found a lesser degree of spalling on two other beams. These apparent failures were not reported in the previous inspection in 2002. Such a failure has significant implications to the load-carrying capacity and long-term durability of this structure.

A feature of the beam design was that the prestressing strand was relatively poorly confined (by today's standards), the secondary reinforcement only partly enclosing it. This deficiency is believed to have contributed to the damage. Lack of enclosing stirrups (which, if present, would normally corrode and crack the concrete before the prestressing steel corroded) also prevented early warning of potential corrosion of the strand. The Hamanatua Stream Bridge is close to the coast and it is likely that the corrosion risk was further increased by atmospheric chloride contamination of the concrete.

Damage like this poses an immediate risk to structural capacity. As well as the obvious aspects of safety and collateral property damage, it has short-, medium- and long-term cost implications:

- the need for immediate preventive action where possible;
- the need for repair and strengthening when damage occurs; and
- the possible reduction in capacity and durability may reduce service life, particularly with anticipated future increases in heavy vehicle loads.

The prestressing steel corrosion on the Hamanatua Stream Bridge was one of the first such cases recorded on a prestressed concrete bridge in New Zealand under normal service conditions. It raised the concern that other prestressed concrete bridges of the same design on the New Zealand roading network may face a similar risk.

The H20-S16-T16 standard design loading (Stirrat & Huizing 1961) used in the Hamanatua Stream Bridge was widely used in the 1960s and early 1970s. A search of Transit New Zealand's Bridge Descriptive Inventory[2] prior to this investigation indicated

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[1] In this report 'prestressed' is synonymous with 'pretensioned'. Post-tensioned prestressed structures were not considered in the investigation.

[2] The Bridge Descriptive Inventory, also known as the Bridge Data System, is a computer database administered by Transit New Zealand. People wanting to access this database should contact Transit New Zealand's National Office.

approximately 92 similar prestressed concrete bridges of this design loading on the State Highway network throughout New Zealand. They comprise about 2.5% of the approximately 3800 bridges on the State Highway network. The number of bridges of this type on the Local Authority roading network is unknown.

Repair and strengthening costs on the Hamanatua Stream Bridge are estimated to be in the order of $400,000. If it is assumed that a third of the other 92 bridges of this type on the State Highway network are in similar condition or at risk of similar deterioration then future potential repair costs are likely to be in the order of ten to fifteen million dollars. The potential repair cost for Local Authority bridges is unknown. Thus the potential short-, medium- and long-term costs associated with this type of deterioration in New Zealand could be rather large, particularly if the risks to life and property as a result of a sudden failure are included. Proactive management of the deterioration may reduce the life cycle cost of these bridges, for example by applying preventive treatments before damage is evident, or by determining the most cost-effective state of deterioration at which to repair the structure.

The key objective of this research was to determine the current and future risk of prestressing steel failure in bridges, in particular prestressed concrete beams constructed in the 1960s and early 1970s. The findings will enable Transit New Zealand and Local Authority bridge practitioners to understand the variability in concrete materials and workmanship better, and make informed and cost-effective decisions regarding the management of these structures, including improving or maintaining their load-carrying capacity. As part of its overall asset management process, Transit New Zealand is developing a bridge replacement programme that will involve identifying all risks associated with its bridges, including environment, vehicle loads, standard designs and condition. The findings of the research relate directly to this programme.

Significant failures of prestressed concrete structures have been reported from Germany (Federation Internationale du Beton (*FIB*) 2003). These were related to a combination of problems including prestressing steel properties, the use of high alumina cement, concrete admixtures containing chlorides or thiocyanates, and poor design and construction practices. Durability problems affecting prestressed concrete bridges have also been encountered in the US, Europe and the United Kingdom. These were primarily caused by de-icing salts initiating corrosion of the prestressing steel and are not directly relevant to this investigation apart from the corrosion mechanism.

1.2 Scope of investigation

The research, carried out in 2005–2006, aimed to:

- identify the factors that contributed to the deterioration on the Hamanatua Stream Bridge;
- assess the current condition and future risk of corrosion in both prestressing and conventional reinforcement on other bridges of similar age and design, in a range of exposure environments in New Zealand, to ascertain whether this type of deterioration is widespread;
- assess the variability in materials and workmanship for this type of bridge beam; and
- develop recommendations for the future management of these structures to assist New Zealand bridge owners and managers to optimise the economic life of the bridge stock and the remaining life of individual structures.

An implicit aim was to find out whether the corrosion risk has been reduced by current prestressed concrete beam designs.

The research involved:

- examining the design of the Hamanatua Stream Bridge;
- searching the Transit Bridge Descriptive Inventory to identify other bridges of similar design on the State Highway network;
- searching for information on international experience with corrosion of prestressing steel and associated failures;
- assessing the condition of the Hamanatua Stream Bridge, including assessing the condition of remaining prestressing strands and measuring concrete contamination levels which may have contributed to corrosion of the steel;
- assessing, in a similar way, the condition of a range of bridges of similar construction type throughout the New Zealand State Highway network that were selected to represent a range of reported conditions and environmental exposure;
- evaluating the feasibility of a simple model for predicting the onset of prestressing corrosion, which could be used by bridge practitioners to optimise maintenance and intervention strategies for these bridges; and
- assessing whether the inherent risks associated with the beam designs and materials used in the 1960s and early 1970s have been addressed by current designs and materials, or whether some of the shortcomings identified in these earlier bridges remain.

The original research proposal was based on reports of failure of prestressing wire on the bridge beams. The investigation revealed that prestressing strands, not wire, had corroded, and that the strand had corroded but not actually broken. This limited the scope of the investigation as described in Chapter 3.

Preliminary findings were presented at the 2006 AUSTROADS Bridge Conference (Bruce & McCarten 2006). This report includes the complete findings, plus recommendations for further work that is necessary to develop a strategy for the management of these bridges, including preventive maintenance, repair and strengthening.

2. Corrosion of prestressing steel – overview

2.1 Background

Corrosion of prestressing steel is relatively rare. Because the cross-section of each prestressing wire or strand is small and the steel is already under significant stress, a much smaller cross-section loss from strand or wire (compared to reinforcing bar) will cause the strand to debond from the concrete and eventually break. In addition, it may corrode without producing outward evidence such as rust staining, cracking or spalling because the tensile stresses that the small cross-section of steel generates in the cover concrete are small. Consequently, the strand or wire may debond or break without warning. If it breaks, it may burst from the concrete where the cover concrete cannot withstand the prestressing forces released by the failure. Once one wire (or strand) breaks, its load is redistributed to others that may not have the residual capacity to sustain the extra load, so the risk to the element increases very quickly.

Management of reinforcement corrosion is usually based on a need to maintain the appearance of the structure, prevent damage or injury caused by spalling concrete, or delay the rate of corrosion, i.e. to maintain serviceability. In contrast, management of prestressing corrosion is directly related to maintaining structural integrity because even a small amount of corrosion may affect the bond and thereby the structural performance.

ACI 222.2R-01 (American Concrete Institute 2005) presents a detailed review of the corrosion of prestressing steels. This chapter summarises the salient points.

Three factors contribute to the corrosion of prestressing wire or strand:

- metal properties (these have the least influence on corrosion resistance),
- the quality of the concrete that surrounds the wire or strand (this has significant influence on corrosion resistance),
- service conditions (these have the greatest influence on corrosion resistance).

These factors are described in Chapters 2.2–2.4.

2.2 Metal properties

The properties of the metal are determined by its chemical composition, thermal treatments applied to achieve the desired microstructure, and the methods used to draw the wire/bar and to relieve stress after drawing.

The extra energy input used to produce their higher yield strength means prestressing steels are generally less corrosion-resistant than the steels used in the reinforcing bar. Nevertheless, the corrosion resistance of any prestressing steel will be satisfactory provided that its mechanical properties and composition, and the process by which the strand or wire is manufactured are optimised. Similarly, any prestressing steel can be susceptible to corrosion if these properties are not optimised. The effects of metal composition, structure and mechanical properties on the corrosion resistance of prestressing steels, including examples of prestressing steel failures, are described by *FIB* (2003).

An increase in carbon content or the presence of other elements known as poisons (such as phosphorus, antimony, tin, sulphides and arsenic), particularly as inclusions at metal grain boundaries, can increase the amount of hydrogen entrapped in the steel lattice, increasing the possibility of hydrogen embrittlement (Novokshchenov 1994, *FIB* 2003). Hydrogen embrittlement may increase the risk of failure if the wire or strand is exposed to a corrosive environment. High strength steels are particularly susceptible to hydrogen embrittlement because of their high carbon content.

Until the 1980s, the rod from which a wire was drawn was pretreated by quenching in a molten lead bath to produce the desired microstructure. After that date, the microstructure was formed by cooling the rod in water and then in air immediately after hot-rolling. This change increased the strength of the steel significantly, but not its ductility.

Prestressing wires and bars are made by one of four processes:
- hot-rolled, stretched and stress-relieved bars,
- quenched and tempered martensitic wires/bars,
- cold-drawn, stress relieved wires/strands, and
- cold-drawn wires.

Corrosion activity, particularly that related to hydrogen embrittlement, is concentrated at the most disturbed and weakest part of the metal structure, i.e. the boundaries of the individual metal grains. The manufacturing process determines the orientation of the grain boundaries. Steel with grain boundaries perpendicular to the direction of applied force is more susceptible to premature failure caused by corrosion than steel with grain boundaries parallel to the direction of applied force. In hot-rolled, and quenched and tempered steels, the grain boundaries are perpendicular to the applied force so these types of steel are generally less corrosion-resistant than cold-formed steels, in which the grain boundaries are parallel to force direction.

The most commonly specified prestressing steels, including those included in the current specification for prestressing steel (AS/NZS 4672.1-2007 (Standards Australia 2007)), are cold-drawn and stress relieved.

Quenched and tempered steels with a martensitic structure are the most susceptible to hydrogen embritttlement because they contain a relatively large amount of free hydrogen and their martensitic structure is highly stressed. Their corrosion resistance can be improved significantly by modifying the steel's chemistry to both reduce its free hydrogen content and allow the hardening and tempering process to be optimised to minimise the martensite content (*FIB* 2003).

Similarly, stress corrosion is not normally an issue with ferritic steels such as cold-drawn stress-relieved materials, but martensitic steels such as those produced by quenching and tempering are susceptible to stress corrosion in the presence of sodium chloride solutions. Brittle failure of prestressing steel is sometimes referred to as hydrogen-induced stress corrosion because of the close relationship between hydrogen embrittlement and stress corrosion cracking.

Hydrogen embrittlement and stress corrosion cracking may promote brittle failure of the prestressing steel, and can accelerate the damage caused by corrosion alone. Current specifications for prestressing steel aim to minimise this risk.

To determine whether the nature of the prestressing steel used in a particular bridge has contributed significantly to an observed failure, three questions need to be answered:

- Was the same steel used in all beams on the bridge? If not, is the poor performance of one beam related to the wire product used?
- Did the failed steel comply with the design specifications for the bridge (i.e. was the correct steel used, or was the product or batch of steel substandard)? If not, the problem may be limited to this bridge or others built with the same product.
- Does the failed steel comply with current specifications for prestressing wire (AS/NZS 4672.1-2007(Standards Australia 2007)), i.e. from current knowledge, would we expect it to perform satisfactorily?

To answer these questions, samples need to be taken on site to determine the tensile strength of the wire or strand, its microstructure, its chemical composition, and whether the strand failed in a brittle or ductile mode. Unless the strand or wire has completely broken and is no longer stressed, this cannot be done without significantly reducing the load-carrying capacity of the structure.

2.3 Concrete quality

Steel in concrete is normally protected from corroding by the cement paste's high alkalinity and its relatively low permeability to moisture, oxygen and chlorides. Corrosion will be initiated if the alkalinity of the cover concrete is reduced by carbonation or if it is contaminated by chlorides.

Sufficient depth and quality of concrete cover is essential to protect the prestressing steel from the ingress of moisture, oxygen and chlorides. Selection of an appropriate mix design is important, but cracks and voids will increase the permeability of even the best concrete mix designs. Significant corrosion damage is more likely when the concrete's permeability is increased by inadequate compaction, inappropriate mix design or insufficient thickness over the steel. Should the prestressing steel fail, deeper cover will reduce the risk of the cover concrete cracking or spalling, and the risk of the strand or wire bursting out of the element.

2.4 Service conditions

Although corrosion can be initiated by loss of concrete alkalinity by carbonation, most cases of prestressing corrosion reported in the literature are related to ingress of moisture and chlorides caused by poor drainage. This poor drainage results from poor design and poor maintenance of features such as drains and joints.

Chlorides are a particular problem because they can cause very localised corrosion pitting, which may reduce the cross-section sufficiently to cause the steel to fail under a normal working load. Acidification of corrosion pits may lead to hydrogen embrittlement. In addition, corrosion may be promoted at lower chloride concentrations than for unstressed steel. Steel may be contaminated with chlorides before being cast into the concrete, e.g. by storage on site in a marine environment, or it may be contaminated during the service life of a structure exposed to seawater, sea spray or de-icing salts. Chlorides may also be introduced into the concrete at the time of construction in the form of accelerating admixtures based on calcium chloride, which were sometimes used in precast concrete until the 1980s.

Stray currents from electrical or cathodically protected services may also induce corrosion. Corrosion induced by stray currents is easily detected by a characteristic appearance.

Overloading can cause the premature failure of a wire undergoing general corrosion. Pure overloading is usually characterised by a ductile failure mode, although the relatively lower ductility of prestressing steels may make a ductile failure difficult to detect.

In addition to hydrogen embrittlement and stress corrosion, corrosion fatigue and fretting corrosion may occur, particularly in partially prestressed elements (Nurnberger 2002) or where the bond to the wire/strand has been lost, e.g. by corrosion of the strand surface.

2.5 Assessment of prestressed elements

Because of the risks involved and because outward signs of corrosion in prestressing steel are often absent, inspection of prestressed structures may need to be more rigorous than for reinforced structures. Features that may indicate an increased likelihood of corrosion on prestressed elements include:

- drainage of runoff over the surface;
- cracking, particularly if not expected from normal loading;
- insufficient depth of concrete cover, particularly on surfaces exposed to runoff or chloride ingress;
- physical damage that reduces the effective cover depth;
- leaking deck joints or other features of poor surface drainage that provide a source of moisture for corrosion;
- inadequate concrete consolidation, as evidenced by surface voids;
- reduced alkalinity of concrete cover, particularly on surfaces exposed to runoff; and
- elevated chloride ion content in cover concrete.

Design drawings will identify factors that may affect the corrosion risk. As-built drawings are particularly useful for identifying details and materials, as are ordering or purchase records of materials.

Corrosion of steel in concrete is often related to the presence of air voids at the steel surface, so good quality concrete and good compaction is particularly important. In addition, honeycombed or highly porous concrete could allow thinner wires to lose a significant proportion of their cross-section with no external evidence because corrosion products can be accommodated in the voids without generating the expansion stresses that would otherwise crack or spall the cover concrete.

Measurement of electrochemical corrosion potential (also known as 'corrosion potential') has been reported to be effective in detecting areas where corrosion is most likely (Novokshchenov 1997), although it may be difficult to distinguish between the risk to reinforcement and the risk to prestressing steel. Commonly used assessment criteria may not apply to wires or strands with a higher cover than reinforcement, so variations in electrochemical corrosion potential may be more relevant than the absolute values. In a chloride-contaminated environment, the strand/wire's performance may be limited by the risk of localised pitting corrosion, which is not necessarily detected by corrosion potential measurements. Ali & Maddocks (2003) reviewed non-destructive methods of detecting corrosion of prestressing *in situ* and concluded that a combination of techniques is required to assess the condition of prestressing tendons.

3. Methodology

This project sought to find out what had caused the corrosion of the prestressing strand on the Hamanatua Stream Bridge, and whether the same risks were present on the other elements and on bridges of similar design. The intended methodology was to consider whether the following features on the affected beam were common to the other elements and to similar bridges:

- features of the corroded prestressing strand as described in Chapter 2.2: whether they meet the requirements of the original design and of current practice;
- features of the concrete as described in Chapter 2.3: whether they meet the requirements of the original design and of current practice; and/or
- features of environmental exposure and loading conditions as described in Chapter 2.4.

The site investigation revealed that the corroded prestressing strand had not yet broken, so no information about failure modes could be gleaned. In addition, it was too dangerous to sample the unbroken strand to determine its composition. Consequently, the composition and likely failure mode of the prestressing strand could not be considered in this investigation, which then focused instead on determining critical features of the concrete and the service environment.

3.1 Review of the Hamanatua Stream Bridge design

The design of the Hamanatua Stream Bridge was analysed to determine whether it met current requirements of Transit New Zealand's Bridge Manual (TNZ 2003). Deficiencies and their significance were identified.

3.2 Selection of other bridges for investigation

The design of the Hamanatua Stream Bridge having been identified, the Bridge Descriptive Inventory was searched to identify other prestressed I-beam bridges of similar age and to ascertain their distribution. The search criterion used was 'pre-1973 precast pretensioned I-beam bridges'. The search revealed 117 bridges meeting this criterion.

Twenty-nine bridges, or 25% of the 117 bridges of interest, were selected for assessment. These structures were selected on the following basis:

- For convenience, all were in the central North Island, and in general proximity to the Hamanatua Stream Bridge.
- They were from several different regions to ensure that beams from a range of precast concrete yards were sampled.
- They represented a range of exposure classifications as defined by NZS 3101: 2006. The sample included sixteen bridges in the A2 exposure zone (inland environment), five bridges in the B1 zone (coastal perimeter) and eight in the B2 zone (coastal frontage). The bridges in the B2 zone are all 500 m or less from open surf beaches affected at times by onshore winds.

The selected bridges ranged in age between 35 and 48 years at the time of inspection. They included bridge beams prestressed using high tensile wire and prestressing strand with the adoption of these materials following international trends as the technology advanced.

The locations of the selected bridges are shown in Figure 3.1 and described in Table 3.1.

Chloride contamination depths on a thirtieth bridge, the Turihaua Stream Bridge (SH 35, 308/3.76), were measured and compared to those measured 15 years previously, and the effect of this measured increase in contamination on predicted corrosion risk assessed. This bridge was built in 1978, has a HN-HO-72 design loading, has a deck constructed of hollow core units and is in the B2 exposure zone. Although younger and of different design, it was considered to face a similar corrosion risk to the bridge designs on which this investigation is focused.

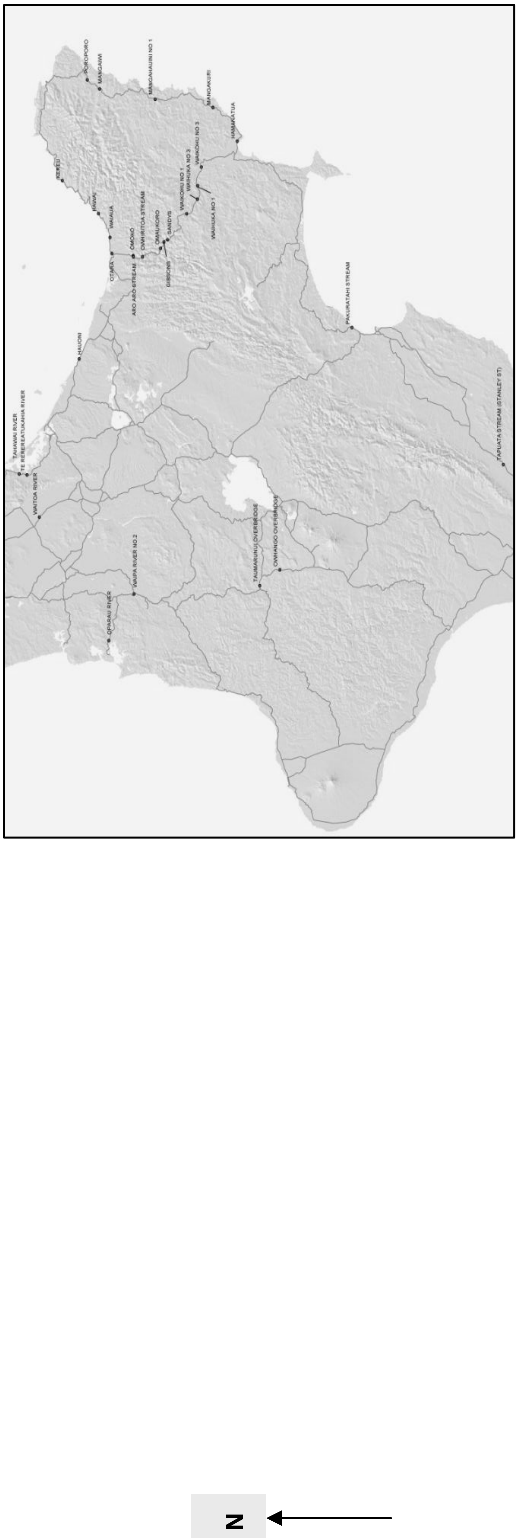

Figure 3.1 Location of bridges selected for inspection related to corrosion.

Table 3.1 Details of bridges sampled for signs of prestressing corrosion.

Bridge name	State Highway	Route position	Location	Distance from coast[a]	NZS 3101 Exposure Zone[a]	Type of Inspection[b]
Tahawai River	2	116/0.00	Northwest of Tauranga	280 m to sheltered harbour	B1	Cursory
Te Rereatukahia Stream	2	116/5.43	Northwest of Tauranga	890 m to sheltered harbour	B1	Cursory
Hauone Stream	2	209/3.01	Northwest of Whakatane	170 m to open surf beach	B2	Cursory
Aro Aro	2	304/12.93	South of Opotiki	14.0 km	A2	Cursory
Omoko	2	318/0.00	South of Opotiki	14.6 km	A2	Cursory
Owhiritoa	2	318/84.10	South of Opotiki	20.2 km	A2	Cursory
Omaukora	2	345/0.00	South of Opotiki	31.3 km	A2	Cursory
Gibsons	2	345/54.80	South of Opotiki	35.7 km	A2	Cursory
Sandy's	2	345/87.60	South of Opotiki	38.7 km	A2	Cursory
Waikohu No.1	2	375/8.00	Northwest of Gisborne	49.9 km	A2	Cursory
Waihuka No.3	2	390/11.76	Northwest of Gisborne	36.0 km	A2	Cursory
Waihuka No.1	2	390/3.33	Northwest of Gisborne	43.7 km	A2	Cursory
Waikohu No.3	2	406/0.77	Northwest of Gisborne	32.0 km	A2	Detailed
Pakuratahi Stream	2	626/5.56	Napier	250 m to open surf beach	B2	Cursory
Tapuata Stream	2	772/4.25	North of Dannevirke	45.0 km	A2	Cursory
Taumarunui Rail Overbridge	4	70/1.83	Taumarunui	59.5 km	A2	Cursory
Owhango Rail Overbridge	4	77/13.60	Southeast of Taumarunui	72.4 km	A2	Cursory
Waitoa River	27	46/6.72	East of Morrinsville	24.8 km	A2	Cursory
Oparau	31	31/13.93	East of Kawhia	340 m to sheltered harbour	B1	Detailed
Waipa	31	53/4.25	Otorohanga	28.3 km	A2	Cursory
Otara	35	0/1.22	East of Opotiki	1.4 km to open surf beach	B1	Cursory
Waiaua River	35	11/0.00	East of Opotiki	270 m to open surf beach	B2	Cursory
Poroporo	35	180/0.00	North of Gisborne	6.0 km	B2	Cursory

Table 3.1 (cont.) Details of bridges sampled for signs of prestressing corrosion.

Bridge name	State Highway	Route position	Location	Distance from coast[a]	NZS 3101 Exposure Zone[a]	Type of Inspection[b]
Mangaiwi	35	190/0.00	North of Gisborne	7.2 km	A2	Cursory
Mangahauini No.1	35	238/0.00	North of Gisborne	100 m to open surf beach	B2	Detailed
Hawai River	35	28/0.00	East of Opotiki	100 m to open surf beach	B2	Detailed
Mangakuri	35	289/4.29	North of Gisborne	4.3 km	B1	Detailed
Turihaua	35	308/3.75	North of Gisborne	150 m to open surf beach	B2	N/A
Hamanatua	35	321/0.00	Gisborne	200 m to open surf beach	B2	Comprehensive
Kereu	35	65/7.91	Northeast of Opotiki	460 m to open surf beach	B2	Comprehensive

Notes to Table 3.1:
a See Chapter 3.4
b See Table 3.2.

3.3 Concrete condition assessments

3.3.1 Types of inspection

The selected bridges were subjected to three different levels of assessment as described in Table 3.2. Elements on the structure were identified as described in Chapter 3.3.2. Each assessment included a visual inspection as described in Chapter 3.3.3. The detailed and comprehensive assessments also included measurements of the properties described in Chapters 3.3.4–3.3.8.

Bridges representing a range of locations and exposure zones were selected for each assessment type.

Table 3.2 Levels of assessment used on bridges in this study.

Level	Looked for:	Measured:	Sample size	Time taken	No. of bridges
Cursory	cracks spalls exposed reinforcing poor compaction	–	–	1 hr	22
Detailed	cracks spalls exposed reinforcing poor compaction	cover depth compressive strength chlorides carbonation porosity	1–2 beams	½ day	5 2 x B2 2 x B1 1 x A2
Comprehensive	cracks spalls exposed reinforcing poor compaction	cover depth compressive strength chlorides carbonation porosity	≤6 beams	1 day	2 2 x B2

3.3.2 Nomenclature used in inspections

On the Hamanatua Stream Bridge, the abutments and piers were labelled A–D from north to south in the direction of increasing State Highway route distance, and the three spans identified accordingly as AB, BC and CD. On the other bridges, the spans were identified by compass bearings based on the general orientation of the carriageway.

Beams on all bridges were identified as upstream, inner and downstream.

3.3.3 Visual inspection

The beams were visually inspected from ground level by the naked eye and, where appropriate, with binoculars. On most bridges, all beams could be inspected, although the width of the river channel on some bridges prevented full inspection of beams in the central spans.

The inspection concentrated on identifying defects likely to influence the durability of the beams, such as cracking and spalling caused by prestressing or reinforcing steel corrosion, exposed prestressing or reinforcing steel, and poor concrete compaction.

Defects and features of interest were photographed.

3.3.4 Volume of permeable voids

An important influence on the durability of prestressed or reinforced concrete is the ingress of moisture. Water is required to initiate reinforcement corrosion and also acts as the carrier for aggressive agents such as chloride ions. The amount of moisture that can enter the concrete is related to its pore volume and continuity, so the volume of permeable voids (VPV) is a useful durability indicator.

Two concrete cores, nominally 54 mm in diameter, were removed from the web of each beam sampled. The nature and quality of concrete in the cores was described, and the VPV measured. Testing was in accordance with AS1012.21: 1999 (Standards Australia 1999), except that measurements were made on whole cores rather than slices and drying was carried out at 60°C rather than 110°C. This approach allowed the compressive strength of the cores to be measured after the VPV testing.

The quality and likely durability of the concrete in these bridges was assessed by comparing the VPVs measured with limits proposed by Andrews-Phaedonos (1997). His limits are, however, based on drying specimens at 110°C.

3.3.5 Compressive strength

After measurement of VPV (see Chapter 3.3.4), the two cores from each beam sampled were dry-conditioned for seven days in accordance with the NZS 3112: Part 2: 1986 then tested for compressive strength. Testing was in accordance with NZS 3112: Part 2:1986 except that the ratio of core diameter to aggregate size was just under 3:1, rather than 4:1 or greater as required by NZS 3112. The relatively small core diameter was considered acceptable for this work, as the results were needed to provide an indication of strength rather than for compliance purposes (see below). All concretes had the same maximum aggregate size so any effect of this deviation from the standard will be consistent for all samples.

Schmidt hammer readings were also taken on each beam tested. This was to enable some comment to be made on the compressive strength of beams that were subject to tests other than compressive strength. It would also indicate differences in the quality of the surface of the concrete on different beams, which may affect chloride ion ingress and carbonation (Chapters 3.3.7 and 3.3.8). Ten readings were taken on each surface tested.

Compressive strength and Schmidt hammer tests were carried out to give an indication of material quality and variability only. For structural assessment in accordance with Transit New Zealand's Bridge Manual (TNZ 2003), a higher sampling rate for both tests is needed. Structural assessment of individual bridges was considered inappropriate for the scope of this investigation.

3.3.6 Depth of cover concrete

The depth of cover concrete was determined using a digital electromagnetic cover-meter. Cover was measured over reinforcing stirrups in the web and in the soffit of the lower flange of each of twenty beams. Cover-meter readings were calibrated against actual cover depths by drilling to expose the reinforcing steel.

Ideally, the depth of cover over the outermost strand at the top and sides of the lower flange would also have been measured because corrosion of these strands would affect the beam's performance significantly. These covers were not measured, however, because of the risk of damaging the strand when exposing it to calibrate the cover readings.

3.3.7 Chloride ion contamination

Drilled powder samples were collected from various depths from the concrete surface on the beam webs to determine the level of contamination from chloride ions. Samples were taken between stirrups and prestressing strands in the upper half of web. As with the cover measurements, chloride profiles on the top and side of the lower flange may have given a more accurate picture of the risk of chloride-induced corrosion at these highly sensitive positions, but chloride contamination was not measured here because of the risk of damaging the closely spaced strands during the sampling process. The chloride contamination on the web was assumed to be similar to that on the lower flange but no attempt was made to validate this assumption.

A surface sample was removed to give an indication of the chlorides available at the concrete surface, then samples were removed at approximately 20 mm increments to a depth of 60 mm. The samples were then ground and analysed by X-ray fluorescence (XRF), and the chloride ion content expressed as a percentage of the dry weight of concrete.

The corrosion of reinforcing steel in chloride-contaminated concrete is a complex process. Its occurrence and rate depend on several factors (e.g. availability of water and oxygen, concrete permeability) in addition to the chloride content of the concrete. Practically, however, the use of a chloride threshold to indicate the likelihood of reinforcement corrosion provides a reasonable estimate of the likelihood of corrosion. The UK Concrete Society (1984) suggests that some risk of corrosion is associated with chloride ion contents exceeding 0.05% by weight of concrete and a high risk of corrosion is associated with levels exceeding 0.15%. These thresholds were used to assess the likelihood of corrosion in this investigation. The threshold may be slightly lower for prestressing steel depending on the specific conditions to which the steel is exposed. For example, it has been suggested that a threshold as low as 0.02% or 0.03% should be used for prestressed concrete because of the higher associated risk and the possibility of stress corrosion (W. Green, pers. comm. May 2007). This is very close to typical background chloride levels and therefore may be overly conservative. Errors related to sampling and accuracy of analytical test methods would also be more significant at such a low level. This investigation therefore considered 0.05% to be the corrosion threshold.

Chloride contents had been measured on samples from the deck units and abutments of the Turihaua Stream Bridge in 1991 as part of a routine maintenance programme. The chloride content of concrete sampled from the deck units was also measured in the current investigation. No other tests were performed on this bridge.

3.3.8 Carbonation depth

The carbonation depth of the concrete was assessed using a phenolphthalein indicator at holes drilled to take samples for chloride analysis and to calibrate the cover readings. Carbonation depth was also measured on the concrete cores removed from the beams.

3.4 Service conditions

The environmental exposure classification for each bridge was determined in accordance with NZS 3101: 2006 by comparing the bridge location with the definitions of exposure classifications C (tidal/splash/spray) and B2 (coastal frontage), and the boundary between exposure classifications B1 (coastal perimeter) and A2 (inland). The classifications thus obtained are listed in Table 3.1.

Specific conditions at each bridge were also taken into account, such as exposure to sea spray. Such exposure conditions are also noted in Table 3.1.

4. Design and distribution of prestressed I-beam bridges built in the 1960s

The 1956 Ministry of Works Bridge Manual set out the criteria for the design and specification of bridge construction in New Zealand and, with subsequent amendments, was the prime reference document until the 1970s, when new standards MWD CDP 701 (Ministry of Works and Development 1972–1978) and NZS 3101P: 1978 (Standards Association of New Zealand 1978) were introduced.

4.1 Beam design

The Public Works Department and the Ministry of Works (MoW) were the early designers and builders of road bridges in New Zealand. Standardised beams and superstructure forms were the norm because they made the design and production processes much quicker and easier. Prestressed I-beam bridges were first constructed in New Zealand in the late 1950s. Standard prestressed bridge beam designs were first issued by the MoW in 1957 and published in 1959. Beam design was based on working stress alone (i.e. serviceability limit state) until NZS3101: 1982 (Standards Association of New Zealand 1982a & b) introduced the need to also consider ultimate limit state.

In the 1950s and early 1960s, beams were designed to the AASHTO H20-S16-44 design loading (AASHO 1944). Typical I-beams were shallow in depth, with six or more beams used per two-lane deck width. They used 0.2 inch (5 mm) diameter pretensioned high tensile wire, with a minimum 0.9 inch (23 mm) cover to the wire in the web and a 1 inch (25 mm) cover to stirrups in the beam soffit. Figure 4.1 shows an example of one of these 'first generation' bridges. Figure 4.2 shows the configuration of the prestressing wire and stirrups used in this design.

Figure 4.1 Example of a bridge of the H20-S16-44 design (AASHO 1944).

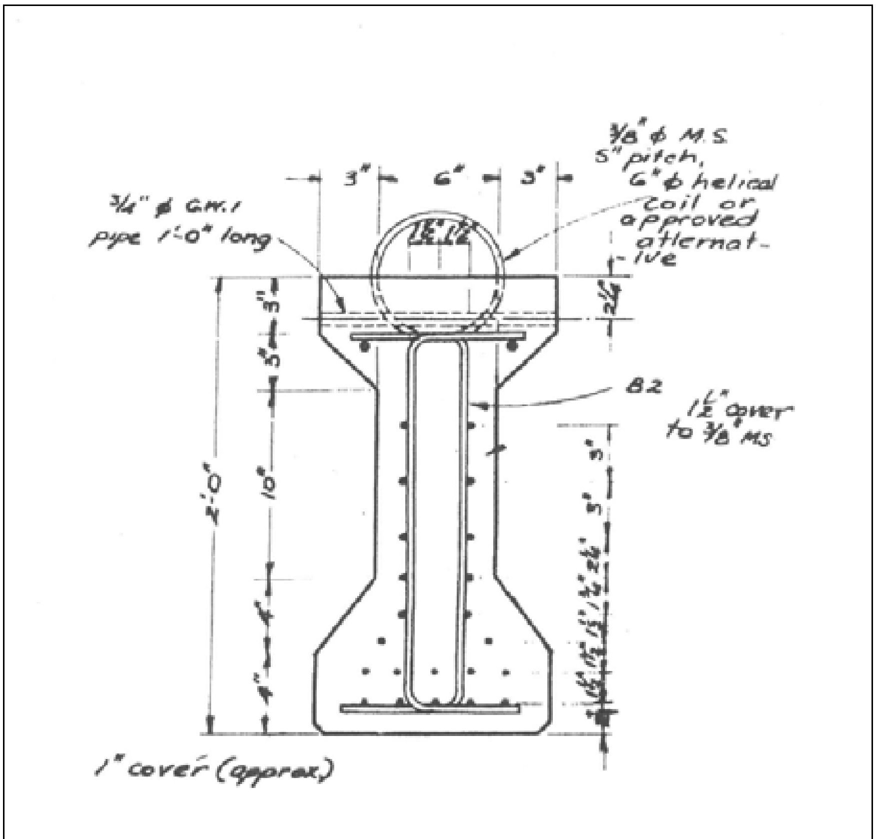

Figure 4.2 Configuration of stirrups and prestressing wire, H20-S16-44 design (AASHO 1944).

With the introduction of AASHTO H20-S16-T16 in 1961 (Stirrat & Huizing 1961), a 'second generation' of standard bridge beams was developed. These I-beams were deeper, generally with four or five beams per two-lane deck width. They used 3/8 inch (9.5 mm) diameter stress-relieved strands, with a minimum 1 inch (25 mm) cover to the stirrups in the web and beam soffit, and an 11/8 inch (29 mm) cover to the strand. This design was used, with a number of modifications and improvements, through to the early 1970s. An example of the design for a nominal 45 foot span is shown in Figure 4.3.

Some bridges were originally designed to the AASHTO H20-S16-44 design (AASHO 1944) but the beam design was subsequently altered to AASHTO H20-S16-T16 (Stirrat & Huizing 1961). These bridges may have more than four to five beams per span but the beams contain strand rather than wire. Kereu Bridge is an example of this.

In both these designs, the number of prestressing strands varied with span length. For example, in the second generation design, the number of strands in the bottom flange increased for a 50 foot span as shown in Figure 4.4. To further increase the span to 55 feet, as used in the Hamanatua Stream Bridge, the height of the beam was increased and extra strands were added to the now longer web. Fitting extra strands into a flange cross-section that is only slightly deeper than the 45 foot design clearly increases the likelihood of inadequate cover to the strand on the upper corner of the bottom flange. The number of stirrups and their spacing also varied with the span length.

A critical feature of both designs is that the prestressing steel is not fully confined by the stirrups. Apart from the structural implications, this means that where the strand is not confined, the first sign of a corrosion problem on the beam is damage caused by corrosion of the strand itself, which is immediately structurally significant. The only surfaces on which a warning of future strand corrosion damage may be given are where the stirrup lies between the strand and the outer surface, i.e. on the beam soffits; and, on the second generation (post-1965) bridges, the webs. Corrosion of the strands on the side of the beam flanges was indeed the first sign of a problem on Hamanatua Stream Bridge (see Chapter 5). Although the first generation beams may seem to be at greater risk from corrosion because they have more unconfined strand, in practice, the risk is similar for both designs because the most critical strand, on the outer corner of the flange, is unconfined in both designs.

MoW specification MOW 5920 (MoW 1962) specified that all high tensile strand used in such beams be stress-relieved and, where applicable, conform to ASTM A416-57T (ASTM International 1959). AS/NZS4672.1:2007 also requires that strand be stress relieved. The 1999 and 2006 versions of ASTM A416 (ASTM International 1999 and 2006), however, specify low relaxation strand as the norm and that stress-relieved strand must be specifically ordered if required. In practice, the differences between the steel types have less influence than service conditions on corrosion resistance, providing the steel does not have a martensitic structure (see Chapter 2.2).

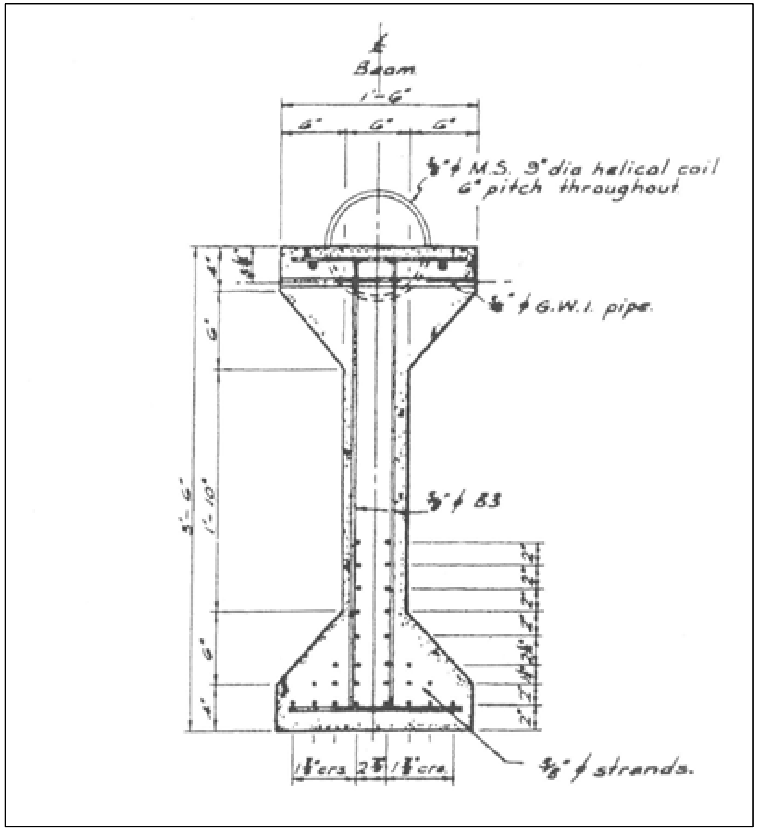

Figure 4.3 'Second generation' H20-S16-T16 design for a 45-foot span (Stirrat & Huizing 1961).

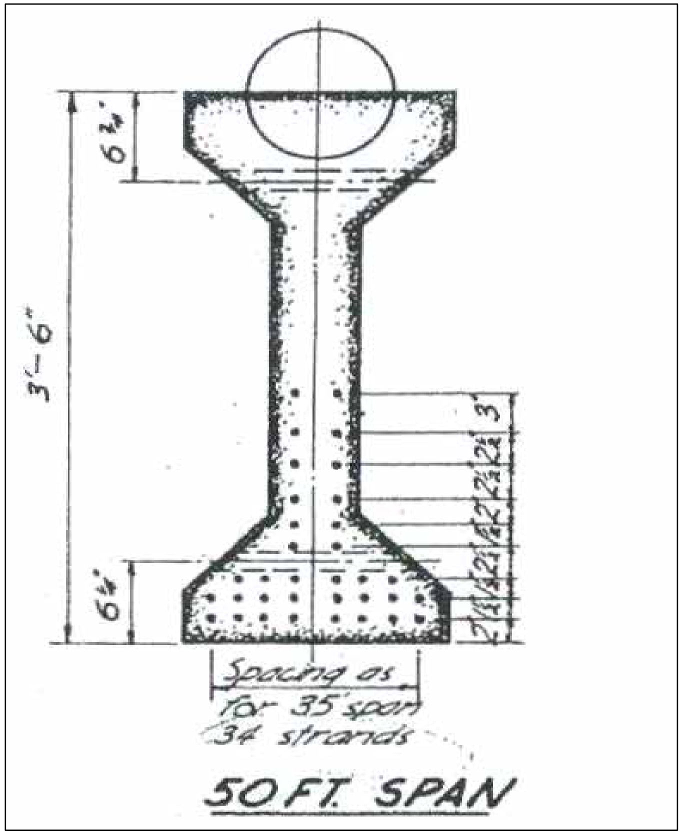

Figure 4.4 'Second generation' design for a 50-foot span.

Notes to Figure 4.4:

(a) Stirrups not shown in this generic drawing.
(b) For a 55′ generic design, the beam height increased to 4′0″, with three extra strands at 2-2.5″ spacing along each side of the web.

Improvements in manufacturing processes have increased the ultimate strength of strand so that the minimum breaking load requirement of AS/NZS4672.1:2007 for a seven-wire 9.5 mm diameter strand is 102 kN compared to the 1959 requirement of 93.5 kN (21 kilo pounds force or 'kips') for a 0.375 inch strand, and proof load forces have increased correspondingly. ASTM A416-06 allows for an additional grade of strand, which is stronger than the original. Newer bridges built with stronger strands will be of more efficient design, containing a smaller area of prestressing steel. This means that the consequences of prestressing steel corrosion in bridges built to current requirements may be higher, irrespective of possible changes in corrosion resistance related to the manufacturing method.

4.2 Concrete quality

In the early 1950s, reinforced concrete exposed to seawater was required to have a 28-day compressive strength of 3500 psi (approx. 24 MPa) and a water to cement ratio (w/c) of 0.5, compared to 3000 psi (approx. 20 MPa) and a w/c of 0.5 for concrete not exposed to seawater. Prestressed concrete was required to have a 28-day compressive strength of 5500 psi (38 MPa). Until the 1980s, calcium chloride admixtures were sometimes used to accelerate early strength development.

Variations in the quality of concrete construction may have contributed to a reduction in concrete durability in some regions. For example, in Gisborne, unwashed beach sand and relatively soft aggregates susceptible to significant moisture movement were used. A study of concrete quality in all bridge elements (Rowe et al. 1986) found that poor compaction was less common in post-1950 bridge beams in Gisborne than in Taranaki or Wellington. Rowe et al. also found that plastic shrinkage in decks was more common in Gisborne than in the other two regions but limited to a period between the mid-1950s to the mid-1960s. Former bridge engineers (A. Watton and A. Tuck, pers. comm. September 2005) advised us that the Hamanatua Stream Bridge beams were not locally made and that they used 'good quality' materials and 'good' construction practices. They also said that from the mid-1960s, concrete aggregates for bridging works in the Gisborne area were required to be sourced from particular quarries, which coincides with the improvements in deck quality observed by Rowe et al.

4.3 Bridge descriptions

Based on the type of prestressing reported in construction drawings, of the 29 bridges examined:

- five (built between 1958 and 1964) were of the AASHO H20-S16-44 design,
- seventeen (built between 1962 and 1971) were of the AASHTO H20-S16-T16 design, and
- seven (built between 1961 and 1971) were of unknown design.

Of the 'unknown' designs, four (built between 1961 and 1964) have seven to nine beams and are probably of the AASHO H20-S16-44 design; three (built between 1966 and 1971) have four beams per span and are probably of the AASHTO H20-S16-T16 design.

The Hamanatua Stream Bridge, built in 1966, is an example of the AASHTO H20-S16-T16 design described in Chapter 4.1. It consists of three spans, each fifty-five feet long with five I-beams and one mid-span diaphragm as shown in Figure 4.5 and Figure 4.6.

Figure 4.5 Layout of the middle span, Hamanatua Stream Bridge.

Design and construction features of all bridges inspected during this investigation are presented in Table 4.1.

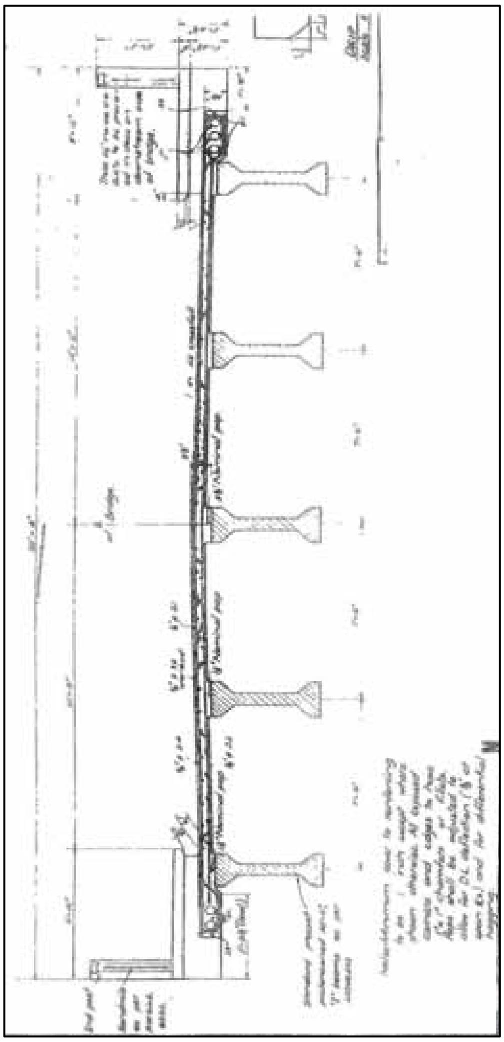

Figure 4.6 Hamanatua Stream Bridge cross-section.

Table 4.1 Design and construction features of bridges inspected.

Bridge name	Date of construction	'Generation'[a]	Number of spans	Beams per span	Prestressing type [b]	Minimum specified concrete cover [c] (mm)	Specified concrete strength [d] (MPa)
Aro Aro	1967	2	3	4	10 mm strands	25	38
Gibsons	1967	2	2	4	10 mm strands	25	38
Hamanatua	1966	2	3	5	10 mm strands	25	38
Hauone Stream	1962	1?	1	8	Unknown	Unknown	Unknown
Hawai River	1969	2	5	4	10 mm strands	25	38
Kereu	1964	1	10	9	10 mm strands	25	Unknown
Mangahauini No.1	1966	2	6	5	10 mm strands	25	38
Mangaiwi	1963	1	2	7	10 mm strands	25	Unknown
Mangakuri	1962	1	2	7	10 mm strands	Unknown	Unknown
Omaukora	1967	2	2	4	10 mm strands	25	38
Omoko	1967	2	3	4	10 mm strands	25	38
Oparau	1971	2?	3	4	Unknown	Unknown	Unknown
Otara	1961	1	7	11	5 mm wires	23	Unknown
Owhango Rail Overbridge	1958	1	1	14	5 mm wires	23	38
Owhiritoa	1967	2	3	4	10 mm strands	25	38
Pakuratahi Stream	1962	1	6	6	10 mm strands	25	Unknown
Poroporo	1969	2	3	4	10 mm strands	25	38
Sandy's	1968	2?	1	4	Unknown	Unknown	Unknown
Tahawai River	1961	1?	3	9	Unknown	Unknown	Unknown
Tapuata Stream	1968	2	3	4	10 mm strands	25	Unknown
Taumarunui Rail Overbridge	1964	1	3	9	5 mm wires	23	Unknown
Te Rereatukahia Stream	1968	2	3	4	10 mm strands	25	38
Waiaua River	1962	1?	8	6	Unknown	Unknown	Unknown
Waihuka No.3	1971	2	3	4	10mm strands	25	38
Waihuka No.1	1962	1	4	9	5 mm wires	23	Unknown
Waikohu No.3	1970	2	6	4	10 mm strands	25	38
Waikohu No.1	1960	1	3	9	5 mm wires	23	Unknown
Waipa	1964	1?	9	7	Unknown	Unknown	Unknown
Waitoa River	1966	2?	3	4	Unknown	Unknown	Unknown

Notes to Table 4.1:

a Based on prestressing type, except where indicated by '?', where generation is based on number of beams. First generation bridges are built to AASHO H20-S16-44 design; second generation to AASHTO H20-S16-T16.

b 10 mm prestressing strand was specified as 3/8 inch diameter stress relieved strand; 5 mm wire was specified as 0.200 inch diameter pretensioned high tensile wire.

c Specified concrete cover over outermost steel in web. Specified as 0.9 inch in the first generation design and 1 inch in the second generation design.

d Concrete specified as 5500 lbs/in.

4.4 Distribution of 1960s prestressed I-beam bridges

The 117 'pre-1973 precast pretensioned I-beam bridges' identified from the Bridge Descriptive Inventory are distributed throughout the country as shown in Figure 4.7. Seventy-three of these bridges are in the North Island, 46 of which are within 10 km of the coast; 44 bridges of these bridges are in the South Island, 16 of which are within 10 km of the coast.

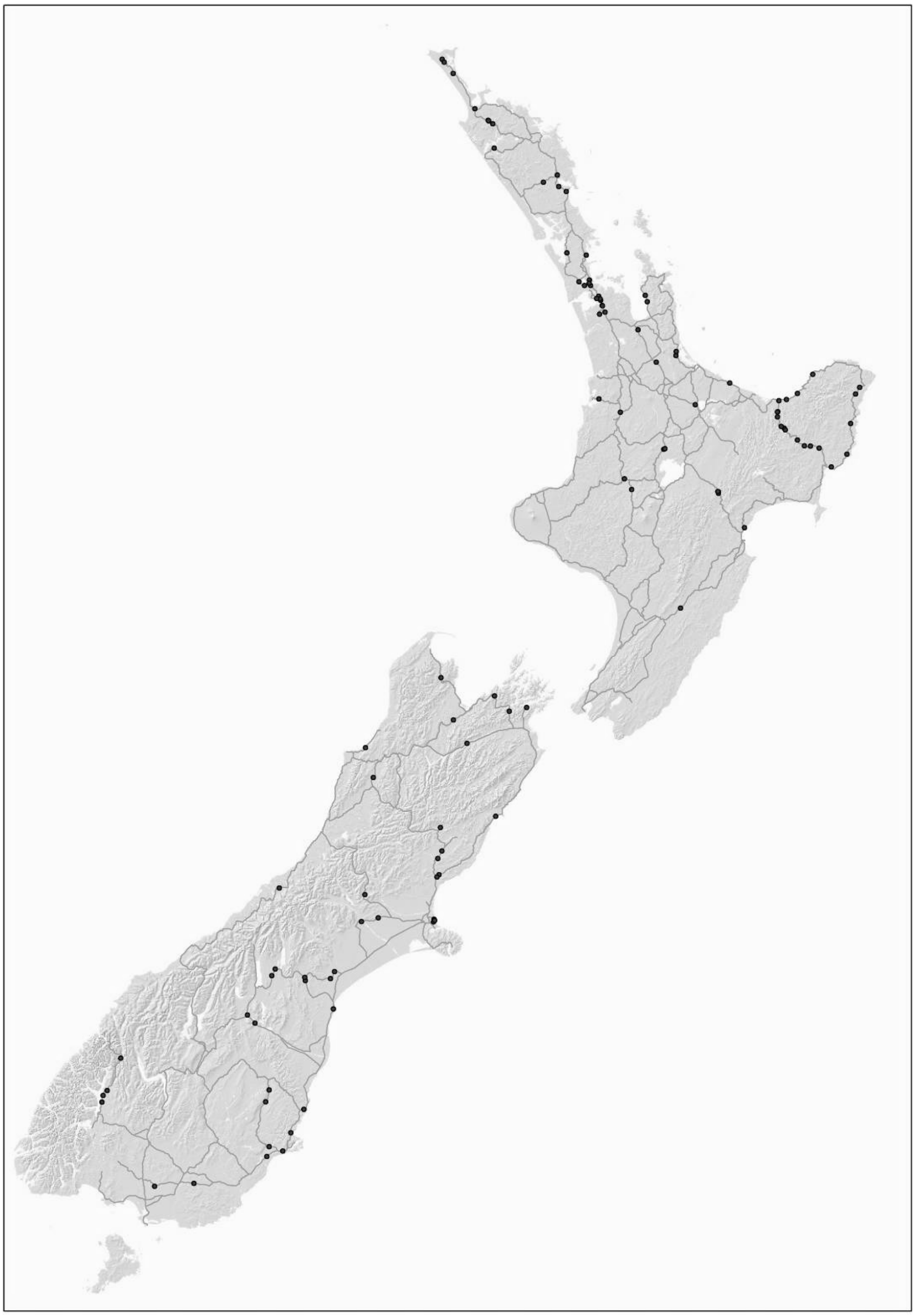

Figure 4.7 Distribution of pre-1973 precast pretensioned I-beam bridges on New Zealand State Highways.

5. Observations and test results

5.1 Bridge condition

The principal defect on the Hamanatua Stream Bridge beams is corrosion of the prestressing strand in the side face of the bottom flange.

The worst affected area is at the north end of the upstream beam in Span BC, where two prestressing strands are exposed and corroding over a 2.5 m length (Figure 5.1). The loss in strand cross-section was estimated to be about 10%. Cracking and spalling extends from this area through to the mid-span of the beam. In the worst affected area, cover to the prestressing steel at the side of the beam flange exceeds 30 mm and is virtually the same as the cover to reinforcing stirrups in the soffit of the flange. Although both the prestressing steel and the stirrups are corroding, the cracks and spalls are over prestressing strands rather than stirrups. This may be because of the higher corrosion expansion forces exerted by the closely spaced strands in this area, or may be related to movement of the cover concrete when the bond to the strand failed and the prestress was lost.

Figure 5.1 Spalling over prestressing strand on the Hamanatua Stream Bridge, Span BC.
Note:
The black arrows point to the strand. The white arrows show the ends of the stirrups, which do not fully encase the strand.

The bottom flange of one inner beam in each of Spans AB and BC are also cracked along lengths of up to 500 mm along the line of the strand (Figure 5.2). The prestressing steel is currently not exposed but the cracks will eventually develop into more significant spalls.

Figure 5.2 Cracking over a prestressing strand on the Hamanatua Stream Bridge.

None of the other bridges inspected exhibited cracking or spalling caused by prestressing steel corrosion. Slight corrosion on a strand exposed by a spall caused by impact damage was observed on Hawai River Bridge (Figure 5.3). Although Transit New Zealand’s bridge inspection records for Kereu Bridge indicate prestressing strands were exposed and corroding in some spans, only corrosion of conventional reinforcing steel was observed in this investigation (Figure 5.4).

Figure 5.3 Corroding strand on Hawai River Bridge. This spall was caused by impact damage rather than corrosion.

Figure 5.4 Spalling associated with low cover over a stirrup on the beam soffit, Kereu Bridge.

Eight of the inspected bridges exhibited damage caused by corroding reinforcing steel.

On five of these bridges, minor spalling has occurred in the bottom flange of some beams as a result of poor placement of individual reinforcing stirrups. On these beams, the corrosion was related to low concrete cover in the soffit of the beam flanges and over the cut ends of the stirrups in the side of the flanges (Figure 5.5); cover depths were as low as 5 mm in some instances. These spalls were generally isolated, with only one or two affecting each bridge, and they are of no significant durability concern.

On three of these bridges, all in coastal exposure zones, the beams were affected by widespread spalling in the bottom flanges of the beams, resulting again from low cover to the stirrups as shown in Figure 5.4. Lack of cover in these beams has implications for the long-term durability of prestressing steel adjacent to the stirrups if the cover to the strand is also lower than the design value. The beams on these bridges are characterised by numerous nails and tie wires corroding in the soffits, indicating poor quality control at the time of construction, and therefore are likely to have been supplied from the same precast yard. Many of the spalls have been repaired with a cement-based mortar and most have subsequently failed. Repairs to these spalls have been ongoing.

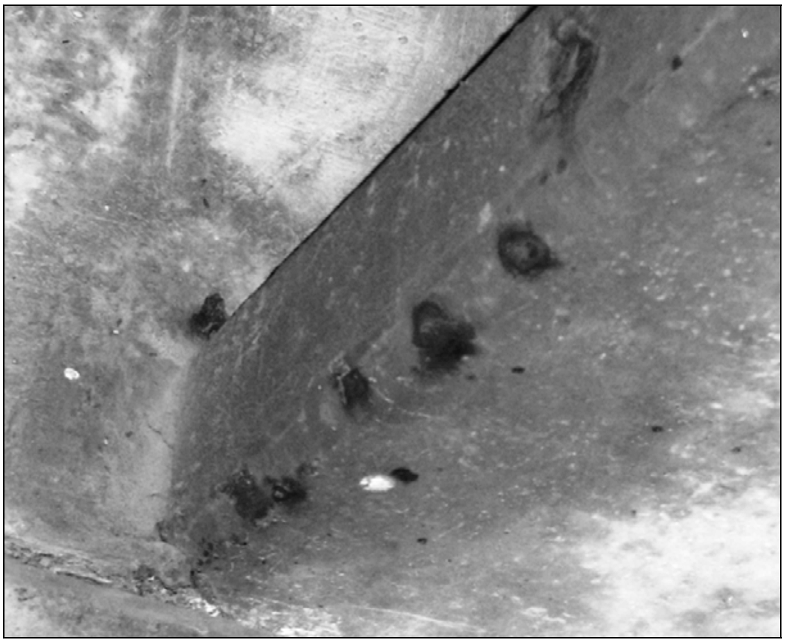

Figure 5.5 Corroding stirrup ends close to surface on Hawai River Bridge.

5.2 As-built concrete quality

Apart from the reinforcement corrosion issues described above, the general impression of the precast beams in the bridges inspected is that they represent good quality precast construction. Minor surface defects, primarily bugholes and areas of grout loss, affect most beams but, in many cases, were bag-rubbed at the time of construction. Shallow areas of poorly compacted concrete were also detected on a number of bridges. A more significant example of poor compaction was detected on Sandy's Bridge, where a third to half the length of two beams was affected to a depth of up to 20 mm (Figure 5.6). The inland location of this bridge means this defect is unlikely to reduce the durability of the concrete, but a similar defect would be more significant on a bridge near the coast.

Figure 5.6 Poor compaction on beam, Sandy's Bridge.

Table 5.1 presents descriptions of the concrete and results from measurements of the compressive strength and VPV made on the core samples taken from the seven bridges subject to detailed or comprehensive inspections.

The core samples revealed that the concrete used in the precast beams was generally well-proportioned and well compacted. The coarse aggregates used were rounded greywacke in the Gisborne and East Cape areas, crushed volcanic aggregate in the Bay of Plenty and crushed greywacke aggregate at Kawhia (Oparau Bridge) on the west coast. The maximum aggregate size in all concretes was approximately 20 mm.

The compressive strength of the concrete cores ranged from 40 MPa to 81.5 MPa. Only one core was weaker than 50 MPa. This low (40 MPa) strength, measured on one beam on Kereu Bridge, means it is unlikely this beam would have met the 28-day specified strength of 38 MPa. All other concretes are likely to have met this 28-day strength requirement.

Average Schmidt rebound numbers on the surfaces from which the cores were taken ranged from 46 to 55. Schmidt rebound numbers are a measure of surface hardness, variations in which, for a given concrete, can indicate local deficiencies in concrete quality or surface finish. They did not always correlate well with the core compressive strengths and therefore are not considered in the context of strength. The similarity of results over all structures reflects their generally good compaction and uniform surface finish.

Transit New Zealand requires its structures to have a specified intended life of 100 years. For structures with a specified intended life of 100 years and made from concrete containing Type GP cement only (i.e. no supplementary cementitious materials added), NZS 3101: 2006 does not permit cover depths less than 30 mm for structures in the A2 and B1 zones, or less than 35 mm for structures in the B2 zone. NZS 3101: 2006 requires a 28-day concrete compressive strength of 60–100 MPa for structures with these minimum covers. The concretes in the bridges assessed would not meet these compressive strength requirements let alone the higher strengths that would be necessary to provide adequate durability at covers less than 30 mm.

The VPV results range from 7.8% to 10.7%. VicRoads requires a VPV less than 14% for coastal/marine structures, less than 16% for structures in the B2 exposure zone and less than 17% for inland structures (Andrews-Phaedonos 1997). This suggests that the VPVs recorded in this investigation are relatively low and therefore that the void structures in the concretes examined here are likely to enhance rather than detract from the overall durability performance. The VicRoads requirements, however, are based on VPVs measured at 28 days after drying at 110°C rather than VPVs measured many years later after drying at 60°C. Consequently, the results from the cores assessed in this investigation may suggest the concrete quality is better than it actually is. Two factors contribute to this. Firstly, the cement would be hydrated to a greater degree after 30 to 40 years than after 28 days. Secondly, the less rigorous drying regime may have resulted in the concretes being wetter when tested and therefore able to absorb less water than they would have been if dried at 110°C (see Chapter 3.3.4). Nevertheless, the VPVs recorded do not indicate that the concrete quality is inadequate.

No correlation between VPV and compressive strength was observed.

Table 5.1 Compressive strength and volume of permeable voids (VPV).

Bridge name	Span	Beam	Concrete description	Compressive strength [a] (MPa)	Schmidt rebound number [b]	VPV[a] (%)
Hamanatua	AB	Inner	20 mm rounded greywacke aggregate. Well-proportioned. Well compacted apart from occasional 1–2 mm air voids.	61.5	51	8.0
		Downstream	20 mm rounded greywacke aggregate. Well-proportioned. Well compacted apart from occasional 1–2 mm air voids.	56.0	51	8.4
	BC	Upstream	20 mm rounded greywacke aggregate. Slightly deficient in fine aggregate. Well compacted apart from occasional 1–2 mm air voids.	60.5	55	9.6
		Downstream	20 mm rounded greywacke aggregate. Well-proportioned. Well compacted apart from occasional 1–3 mm air voids.	56.5	51	10.2
Hawai River	South	Downstream	20 mm angular volcanic aggregate. Well-proportioned. Well compacted apart from occasional 2–8 mm air voids.	81.5	50	8.2
Kereu	South	Upstream	20 mm well-rounded greywacke aggregate. Well–proportioned. Well compacted. Wood present.	40.0	49	7.8
		Downstream	20 mm rounded greywacke aggregate. Well-proportioned. Well compacted.	54.0	50	8.3
Mangahauini No.1	Third from north end	Inner	20 mm rounded greywacke aggregate. Slightly deficient in fine aggregate. Well compacted apart from occasional 1–2 mm air voids.	57.5	–	10.7
Mangakuri	South	Inner	20 mm rounded greywacke aggregate. Deficient in fine aggregate. High proportion of larger aggregate. Well compacted apart from occasional 1–11 mm air voids.	60.5	49	7.8
Oparau	Northwest	Downstream	20 mm angular greywacke aggregate. Aggregate shape is platy. Well proportioned. Well compacted apart from occasional 1–3 mm air voids.	75.5	47	9.4
Waikohu No.3	North	Downstream	20 mm sub-angular volcanic aggregate. Slightly deficient in fine aggregate. Well compacted apart from occasional 1–3 mm air voids.	70.0	46	9.1

Notes to Table 5.1:
a Average of two cores.
b Average of 10 readings.

5.3 Depth of cover concrete

Overall cover depths measured over stirrups on each bridge are presented in Table 5.2. Results from the individual beams on each bridge are presented in Table 5.3.

Table 5.2 Summary of cover depths to stirrups.

Bridge (exposure classification)	Location on beam	Concrete cover to stirrups (mm)[a]		
		Average	Max	Min
Hamanatua (B2)	Web	30	39	21
	Soffit	36	44	30
Hawai (B2)	Web	23	31	14
	Soffit	38	47	28
Kereu (B2)	Web	38[b]	48[b]	29[b]
	Soffit	28	41	14
Mangahauini No.1 (B2)	Web	28	36	18
	Soffit	27	34	20
Mangakuri (B1)	Web	33	45	23
	Soffit	65	90	33
Oparau (B1)	Web	31	38	23
	Soffit	34	37	31
Waikohu No.3 (A2)	Web	29	34	24
	Soffit	36	42	29

Notes to Table 5.2:
a Cover depths less than the specified 25 mm are shaded grey.
b Kereu Bridge is a 'first generation' design, with strands in the web lying outside the stirrups. Cover to the strands on the webs is approximately 10 mm less than the cover to the stirrup.

Average cover depths to stirrups on all beams ranged from 23 mm to 69 mm, but were typically between 30 and 38 mm. Minimum cover depths ranged from 14 mm to 33 mm.

Although the average depths of concrete cover to the stirrups on most beams met the specified minimum of 25 mm, covers to several individual reinforcing bars did not. If the original cover requirement is interpreted as an absolute minimum then the concrete covers to the stirrups in these beams did not meet that requirement.

On Kereu Bridge, the specified cover to the strand on the web is approximately 32 mm. The minimum cover to the stirrups on two of the three beams on which it was measured indicates that minimum cover to the strand was significantly less than 32 mm.

NZS 3101: 2006 defines minimum concrete cover depths for durability based on environmental exposure and specified compressive strength. Using the durability design approach of NZS 3101: 2006, and assuming a specified intended life of 100 years and a concrete compressive strength of 40 MPa, the cover depths required for these prestressed I-beams today would be as follows:

- A2 (inland) 35 mm,
- B1 (coastal perimeter) 40 mm,
- B2 (coastal frontage) 50 mm.

The specified cover of 25 mm and the measured cover depths are clearly deficient when compared to these requirements.

Cover to the prestressing strand is unlikely to vary from that specified because the strand is attached to the formwork in the designed positions and tensioned before concrete is placed. The construction industry has previously expressed concern that cover may be compromised by the concrete ‘sagging’ in the bottom flange as a result of inadequate compaction. This would be evidenced by unevenness, laitance or bubbles on the upper surface of the lower flange, or by plastic settlement cracking at the internal corner between the web and the flange. No such features were observed on the beams inspected; therefore, it was assumed that the cover to the strand was in accordance with the beam design.

Table 5.3 Concrete cover depths measured over stirrups.

Bridge name (exposure classification)	Span	Beam	Location on beam	Concrete cover to stirrups (mm)[a]		
				Average	Max	Min
Hamanatua (B2)	AB	Upstream	Web	28	34	21
			Soffit	36	40	30
		Inner	Web	35	39	30
			Soffit	34	38	32
		Downstream	Web	29	30	26
			Soffit	36	39	32
	BC	Upstream	Web	27	32	24
			Soffit	37	39	35
		Inner	Web	32	37	26
			Soffit	37	44	29
		Downstream	Web	31	33	27
			Soffit	33	40	27
Hawai (B2)	South	Inner	Web	23	27	14
			Soffit	38	47	28
		Downstream	Web	23	31	17
			Soffit	38	44	28
Kereu[b] (B2)	South	Upstream	Web	44[b]	48[b]	41[b]
			Soffit	31	41	25
		Inner	Web	32[b]	36[b]	29[b]
			Soffit	20	32	14
		Downstream	Web	39[b]	41[b]	36[b]
			Soffit	32	41	24
Mangahauini No.1 (B2)	Third from north end	Inner	Web	23	26	18
			Soffit	27	34	20
		Downstream	Web	32	36	30
			Soffit	27	31	22
Mangakuri (B1)	South	Upstream	Web	28	32	23
			Soffit	80	90	75
		Inner	Web	36	45	27
			Soffit	49	58	33
		Downstream	Web	34	37	31
			Soffit	67	86	47
Oparau (B1)	Northwest	Inner	Web	30	38	23
			Soffit	35	37	33
		Downstream	Web	31	34	25
			Soffit	33	34	31
Waikohu No.3 (A2)	North	Inner	Web	31	34	27
			Soffit	33	39	29
		Downstream	Web	27	30	24
			Soffit	39	42	34

Notes to Table 5.3:
a Cover depths less than the specified 25 mm are shaded light grey.
b Kereu Bridge is a 'first generation' design, with strands in the web lying outside the stirrups. Cover to the strands on the webs is approximately 10 mm less than the cover to the stirrup.

5.4 Chloride ion contamination and carbonation depth

5.4.1 Measurements

The likelihood of current and future reinforcement corrosion on these bridges is defined by the depth of chloride ion contamination and carbonation relative to the depth of concrete cover. Table 5.4 presents the results of measurements of chloride ion contamination and carbonation depth. A chloride ion concentration of 0.05% is assumed to represent the threshold over which corrosion is possible (UK Concrete Society 1984). Corrosion may also occur when the depth of carbonation reaches the depth of steel.

Table 5.4 Depths of chloride ion contamination and carbonation measured.

Bridge name (exposure classification)	Span	Beam	Location on web	Depth from surface (mm)	Chloride ion content (weight % of concrete)	Maximum carbonation depth (mm)
Hamanatua (B2)	AB	Upstream	Downstream	Surface	0.269	13
				Surface-20	0.162	
				20–40	0.009	
				40-60	0.014	
		Inner	Downstream	Surface	0.265	13
				Surface-20	0.102	
				20–40	0.012	
				40-60	0.008	
		Downstream	Downstream	Surface	0.289	7
				Surface-20	0.163	
				20–40	0.014	
				40-60	0.008	
	BC	Upstream	Upstream	Surface	0.589	15
				Surface-20	0.341	
				20–40	0.033	
				40-60	0.008	
		Inner	Upstream	Surface	0.536	12
				Surface-20	0.229	
				20–40	0.027	
				40-60	0.017	
		Downstream	Downstream	Surface	0.076	12
				Surface-20	0.111	
				20–40	0.033	
				40-60	0.006	
Hawai (B2)	South	Downstream	Downstream	Surface	0.320	1
				Surface-20	0.197	
				20–40	0.054	
				40-60	0.042	
Kereu (B2)	South	Upstream	Upstream	Surface	0.068	1
				Surface-20	0.032	
				20–40	0.023	
				40-60	0.026	
		Downstream	Downstream	Surface	0.230	1
				Surface-20	0.117	
				20–40	0.025	
				40-60	0.032	
Mangahauini No.1 (B2)	Third from north	Inner	Downstream	–	–	1
		Downstream	Downstream	Surface	0.077	1
				Surface-20	0.046	
				20–40	0.004	
				40-60	0.001	
Mangakuri (B1)	South	Inner	Downstream	Surface	0.056	1
				Surface-20	0.032	
				20–40	0.015	
				40-60	0.019	
		Downstream	Downstream	-	-	1
Oparau (B1)	Northwest	Downstream	Downstream	Surface	0.053	1
				Surface-20	0.018	
				20–40	0.019	
				40-60	0.008	
Waikohu No.3 (A2)	North	Downstream	Upstream	Surface	0.071	5
				Surface-20	0.040	
				20–40	0.044	
				40-60	0.049	
Turihaua	South	Downstream	Downstream	Surface	0.547	–
				Surface–20	0.277	
				20–40	0.034	
				40–60	0.026	

5.4.2 Chloride ion contamination

Reinforcement corrosion on the two bridges in the B1 exposure zone is unlikely now or in the future. Apart from at the surface, chloride contamination levels are less than the 0.05% threshold.

On the Waikohu No.3 Bridge, some 32 km inland and the only bridge examined representing the A2 exposure zone, the chloride ion contamination levels were uniformly 0.04% to 0.05% to a depth of 60 mm. On the Hawai River Bridge, similarly high levels of chloride ions were measured at 40–60 mm deep. The Hawai River Bridge is in the B2 exposure zone so is subject to chloride contamination from sea spray, but the similarity of chloride ion contents for 20–40 mm and 40–60 mm suggests that that concrete 40 to 60 mm from the outer surface may be beyond the influence of atmospheric contamination. The concretes in both these bridges contained a volcanic coarse aggregate, had a relatively high compressive strength compared to most of the other bridges and were similar in appearance, and therefore were probably supplied from the same precast yard. The high levels of chloride at 40–60 mm in these two beams suggest that these two concretes contained chlorides that were added to the fresh concrete, probably as a set-accelerating admixture. The Waikohu No.3 Bridge showed no sign of corrosion in the prestressing steel or reinforcement despite the level of chloride contamination. No corrosion of prestressing steel related to chloride contamination was observed on the Hawai River Bridge.

Chloride ion contamination levels on the Hamanatua Stream Bridge show a high level of chloride contamination in the outer 20 mm. Consequently, any reinforcing or prestressing steel in this outer zone is likely to be corroding, although no steel was detected in this zone on the six beams on which cover was measured. Figure 5.7 indicates some corrosion may be possible at depths of up to 29 mm. Concrete cover depths in these beams are as low as 21 mm, so chloride ion contamination is probably the principal cause of prestressing steel corrosion. Ongoing ingress of chlorides from sea spray will increase the likelihood of future corrosion of prestressing and reinforcing steel. The variation in chloride ion profiles between the different beams tested may represent different micro-exposures on the bridge, Span AB (the end span) being more sheltered than Span BC and the downstream beam of Span BC being the one most exposed to rain washing.

Figure 5.8 shows the depths of chloride ion contamination on the other three bridges in the B2 zone: Hawai, Kereu and Mangahauini.

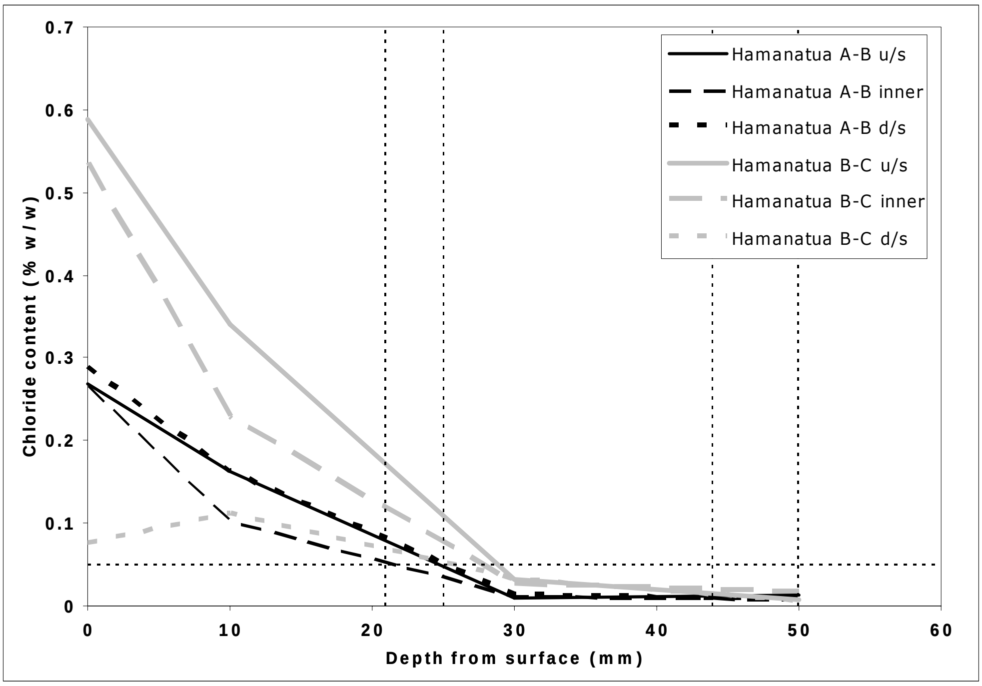

Figure 5.7 Chloride content v. depth from surface on the Hamanatua Stream Bridge.

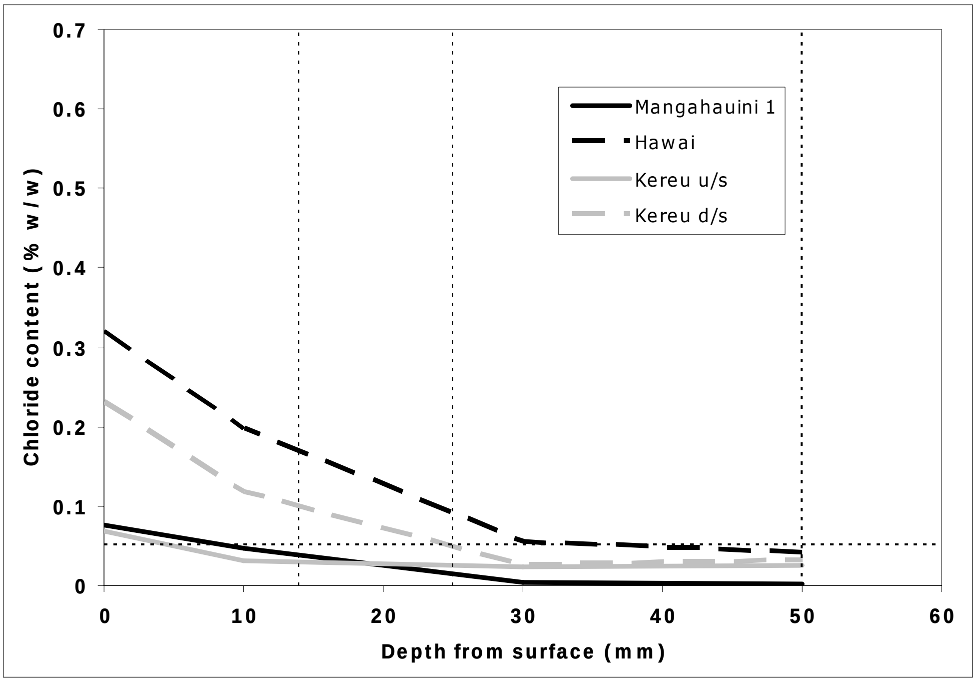

Figure 5.8 Chloride content v. Depth from surface on the Hawai, Kereu and Mangahauini Bridges.

Notes to Figure 5.7 and 5.8:
a The horizontal dotted line represents 0.05% chloride content, which is considered to be the corrosion threshold.
b The vertical dotted line at 14 mm on Figure 5.8 represents the minimum cover depth to stirrups measured on the Hawai Bridge and Kereu Bridge beams. The minimum cover on the Mangahauini Bridge was 18 mm.
c. The vertical dotted lines at 21 mm and 44 mm on Figure 5.7 represent the minimum and maximum cover depths to stirrups measured on the Hamanatua Stream Bridge beams.
d The vertical dotted line at 25 mm represents the specified minimum cover for these beams.
e The vertical dotted line at 50 mm represents the minimum cover required by NZS 3101: 2006 for 40 MPa concrete in a structure in the B2 zone with a 100-year specified intended life.

On the Kereu Bridge, steel with less than the specified 25 mm cover is likely to be corroding because of the presence of chlorides. Indeed, cracking caused by corroding reinforcement was observed on the beams on which the cover and chloride contamination were measured.

On the Mangahauini Bridge, chloride contamination levels at depths greater than 20 mm were below the threshold level. At less than 20 mm, they were close to the threshold. Isolated stirrups close to the concrete surface on the beam webs were corroding.

Chloride ion contamination levels on the Kereu and Hawai River Bridges are similar to those measured on the Hamanatua Stream Bridge, with reinforcement up to a depth of 30 mm likely to corrode because of the presence of chloride ions, particularly steel with less than the 25 mm specified cover. Some conventional reinforcing steel is already corroding on these bridges where covers are particularly low. The chloride contamination levels therefore indicate that the likelihood of current and future corrosion is high.

The measured chloride profiles indicate that chloride induced corrosion is unlikely at a cover depth of 50 mm, the depth specified by NZS 3101: 2006 for concrete for 40 MPa concrete in a structure in the B2 zone with a 100-year specified intended life. This is encouraging for designers building structures in accordance with this standard.

Chloride ion contamination was also measured on the Turihaua Stream Bridge (SH 35, RP308/3.76). The results are included at the end of Table 5.4. The chloride ion profiles are very similar to those recorded on the Hamanatua and Kereu Bridges, which are in similar environments, despite the Turihaua Stream Bridge being 12–14 years younger. Cover depths to the stirrups for beams of this design were 32 mm to the top and soffit surfaces, a minimum of 20 mm to the side in the shear key area and a minimum of 58 mm to the exposed side of the outer unit. Actual cover depths were not measured on the structure during this investigation, or in 1991 when chloride contamination was previously measured. The difference between the 1991 data and the 2006 data is discussed in Chapter 6.3.

5.4.3 Carbonation depths

Carbonation depths in the beams on the Hamanatua Stream Bridge range from 7 to 15 mm. The only other bridge with a carbonation depth greater than 1 mm was the Waikohu No.3 Bridge, which is in the A2 exposure zone.

The ongoing carbonation rate can be estimated using the relationship:

$$X = k.t^{1/2} \qquad \text{[Equation 1]}$$

Where: X = the position of the carbonation front after time t
k = a constant dependant on the porosity of the concrete, the relative humidity and carbon dioxide content of the environment, and the amount of reactable calcium hydroxide in the concrete.

Using this relationship and the measured carbonation depths, the minimum total time for the carbonation front to reach a depth of 21 mm (i.e. the minimum depth of cover) on the Hamanatua Stream Bridge is 78 years from the time of construction (38 years from the time of this investigation). This indicates that carbonation is unlikely to cause reinforcement corrosion now or in the immediate future but may do so in foreseeable life of the bridge.

Using the same method, the carbonation front was calculated to reach the outermost reinforcement on the Waikohu No.3 Bridge in approximately 620 years. Therefore, carbonation on this bridge and on the other five bridges on which carbonation was measured (which have carbonation depths of 1 mm or less) poses no risk to the reinforcement, now or in the foreseeable future.

6. Discussion

6.1 Cause of prestressing strand corrosion on the Hamanatua Stream Bridge

Observations on the beams themselves and on the core samples taken from them indicate that the quality of concrete and workmanship was generally good. Measurements of compressive strength and volume of permeable voids supported these observations. Schmidt hammer tests indicated that the quality of concrete and surface finish on each beam was uniform and was similar on most beams.

Cover depths over stirrups were measured on 12 surfaces: two surfaces on each of six beams (three beams on each of two spans). Average cover depths all exceeded the specified cover of 25 mm, but minimum covers on two surfaces were less than 25 mm and on all but three of the other 12 surfaces, the minimum covers were 30 mm or less.

The bridge is within 200 m of an open surf beach (see Figure 6.1). It is thus highly exposed to chloride ion contamination from sea spray, particularly on surfaces that are not washed by rain. Chloride ion profiles indicated that the chloride ion concentration within 30 mm from the outer surface is high enough to increase the likelihood of steel corrosion. This means that the prestressing and reinforcing steel with less than 30 mm cover is at risk from chloride-induced corrosion.

Figure 6.1 Proximity of the Hamanatua Stream Bridge to open surf beach.

These findings indicate that the corrosion observed was initiated by the low cover depth providing inadequate protection from corrosion induced by chloride ions in the concrete at the surface of the steel. The observed cracking and spalling over the prestressing strand may have been a consequence of the concrete not being able to withstand the expansive stresses imposed by the development of corrosion products on the steel surface. It may also have been facilitated by strain effects associated with loss of prestress caused by failure of the strand-concrete bond.

The cover concrete protects the strand from ingress of chlorides along the length of the beam, but leaking deck joints may increase the risk of corrosion at the beam ends by providing a source of moisture. Staining on the pier caps shown in Figure 4.5 suggests that this may also be a risk on the Hamanatua Stream Bridge, although no evidence of associated corrosion was seen. Once bond is lost between the strand and the cover concrete, water may pass more easily along the steel/concrete interface, further increasing the corrosion risk.

Tensile loads, hydrogen embrittlement, fretting and/or fatigue may eventually cause the strand to break sooner than conventional reinforcing steel corroding at the same rate (see Chapter 6.5). The strand on these beams had not yet broken at the time of inspection, so it was not possible to determine the likely significance of these factors by microscope analysis, the usual means of analysing failure surfaces.

Similarly, the composition of the prestressing steel may have contributed to its corrosion resistance, but without being able to sample a piece of the strand from the structure safely, its composition, and whether it complies with past or current specifications, remains unknown.

Because the strand could not be sampled, more work than originally planned would have been needed to ascertain the influence of stress, hydrogen embrittlement, fretting and/or fatigue on the ultimate failure mode, and to identify whether the prestressing steel used in this bridge had a particularly low corrosion resistance and whether the same prestressing steel was used in all bridges of this design. Once the low cover depths and the high levels of chloride contamination had been identified, it was considered unnecessary to examine these aspects within the current project. Further investigation of these topics may help to optimise mitigation techniques but is not essential for maintaining serviceability.

6.2 Likelihood of prestressing strand corrosion on other bridges of similar design

Corrosion of prestressing steel was not observed on any of the other bridges inspected during this investigation except for minor corrosion of strand on Hawai River Bridge related to impact damage. Exposed and corroding prestressing strands were recorded in Transit New Zealand bridge inspection reports for Kereu Bridge, but only corrosion of conventional reinforcing bars was observed in this investigation.

These observations indicate that cracking and spalling associated with corroding prestressing steel is not widespread, and that routine bridge inspections will probably detect corrosion-induced damage even if the cause is misdiagnosed, e.g. reinforcement corrosion is mistaken for prestressing corrosion or *vice versa*. Any reports of corrosion damage should be followed up with inspection by a suitably qualified and experienced engineer to confirm the cause and determine the associated risk, so an incorrect diagnosis in a routine inspection is not a major concern.

It should be remembered, however, that a 5 mm (0.2 inch) wire could completely corrode without generating enough stress to damage the cover concrete. Thus the absence of visible damage to the concrete cover, particularly in an element exposed to seawater or sea spray, does not necessarily mean that the prestressing steel is in good condition. A method of evaluating the risk from site measurements is therefore needed to detect the likelihood of corrosion before it causes visible damage.

On the bridges where cover depths were measured, the overall average and minimum cover depths were as shown in Table 6.1.

Table 6.1 Overall average and minimum concrete cover depths to stirrups.

Bridge	Overall average cover (mm)	Overall minimum cover (mm)
Hamanatua (B2)	33	21
Hawai (B2)	31	14
Kereu (B2)	33	14
Mangahauini No.1 (B2)	27	18
Mangakuri (B1)	48	23
Oparau (B1)	32	23
Waikohu No.3 (A2)	33	24

Although the average cover depth on all seven bridges was greater than the 25 mm specified cover, the minimum cover on all of them was less than 25 mm. The Hawai, Kereu and Mangahauini No.1 bridges all had minimum covers lower than the Hamanatua Stream Bridge. These three bridges are all in the B2 zone, and within 500 m of open surf beaches. They had similar chloride contamination profiles to Hamanatua, with chloride-induced corrosion likely within 25–30 mm of the surface. Therefore the likelihood of prestressing corrosion is probably similar to that on Hamanatua, and corrosion damage may be imminent or the strand may already be corroding but without having yet damaged the cover concrete. Kereu Bridge, being a 'first generation' design, may be at greater risk than the other three bridges in the B2 zone because the strand in its beams' web is outside the stirrup, and therefore is both unconfined and subject to higher chloride concentrations.

Despite having minimum cover depths similar to Hamanatua, the likelihood of corrosion in the B1 bridges is lower because the concrete in this exposure zone is less exposed to sea spray and therefore the level of chloride ion contamination in the concrete was much lower.

This suggests that corrosion caused by the ingress of chloride ions is likely in bridge beams of this design in the B2 zone (and, by implication, on bridges in the C zone) but unlikely in bridges in the B1 and A2 zones.

Chloride ion profiles measured in two of the bridges suggested, however, that some precasters may have used calcium chloride accelerating admixtures in the concrete to reduce the turnaround time for moulds and/or to allow the beams to be stressed at an earlier age. In the presence of sufficient moisture, steel in concrete containing calcium chloride will be more likely to corrode than steel in concrete not containing calcium chloride, particularly if the concrete is also exposed to external sources of chlorides. Identification of the precaster that produced the beams for the two bridges that were found to contain chlorides from such admixtures will help to identify other bridges with beams from the same source and therefore at similar risk. Measuring chloride profiles on bridges is probably the only way to identify beams from other manufacturers that may be affected. No corrosion was seen on the bridge in the A2 zone that was thought to contain calcium chloride in the concrete, so the risk to such bridges in the A2 and B1 zones may still be low. The risk to bridges in the B2 zone will, however, be significant.

The results from the Turihaua Stream Bridge, representing a double hollow core HN-HO-72 design, suggest that bridges of this design in the B2 zone may also be at risk from chloride-induced corrosion if any reinforcing or prestressing steel has a cover depth less than 28–30 mm from the surface (see Chapter 6.4). Further investigation is needed to find out how the overall risk to these bridges compares to the risk to the pre-1973 bridges.

6.3 Likelihood of future prestressing and reinforcing steel corrosion

6.3.1 Mathematical predictions

The level of chloride contamination in the concrete will continue to increase for as long as the concrete surface is exposed to seawater or sea spray. For example, Table 6.2 shows a significant increase in chloride contents in the outer 30 mm of concrete between 1991 and 2006 on the Turihaua Stream Bridge.

Table 6.2 Chloride ion profiles, Turihaua Stream Bridge.

1991		2006	
Depth from surface (mm)	**Chloride ion content (weight % of concrete)**	**Depth from surface (mm)**	**Chloride ion content (weight % of concrete)**
0–15	0.056	0–5	0.547
15–30	0.056	5–20	0.277
30–45	0.028	20–40	0.034
45–60	0.028	40–60	0.026

Mathematical models have been developed to predict the time at which the chloride content of the concrete at the depth of interest (e.g. specified cover depth) will reach the threshold level at which corrosion can be initiated, also known as the 'time to corrosion initiation'. This only predicts when conditions in the concrete will be such that corrosion may begin, not when it *will* begin, and not when physical damage such as slipping of wire/strand, bond loss, cracking or spalling occurs. The most commonly used corrosion initiation models are based on Fick's 2nd law of diffusion (Equation 2) and Crank's solution to it (Equation 3):

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \qquad \text{[Equation 2]}$$

$$C_{(x,t)} = C_i + (C_s - C_i)\left(1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right) \qquad \text{[Equation 3]}$$

Where:

$C_{(x,t)}$	=	the chloride-ion concentration at depth x and time t, mass %,
C_s	=	the projected surface chloride-ion concentration, mass %,
C_i	=	the initial chloride concentration in the concrete,
x	=	the depth below the exposed surface to the middle of the layer, in metres,
D	=	the apparent chloride-ion diffusion coefficient of the concrete, m^2/s,
t	=	the exposure time
erf	=	the Gaussian error function

This approach and commercially available models based on it are described by Lee & Chisholm (2005). Used in its simplest form, i.e. solving equation 3, it introduces several assumptions that reduce the accuracy of the prediction:

- Diffusion is the only mechanism of chloride ingress.
- Concrete properties are uniform throughout the placement.
- Corrosion is initiated when a critical chloride ion concentration is exceeded at the steel surface, ignoring other aspects of chemistry that may affect corrosion initiation.
- Exposure is uniform over the entire surface of an element.
- Chloride ingress is independent of applied stress.
- A particular concentration of chloride ions is present at the concrete surface.
- Concrete properties change with time at a particular rate.

Statistical methods have been proposed to manage the natural variability in the input values (e.g. Khatri & Sirivivatnanon (2004), Polder & de Rooij (2005), and Zhang & Lounis (2006)). In most practical applications, however, sufficient information about the variability is unlikely to be available. Therefore, instead of using Equation 3 to predict a precise time to corrosion initiation, it may be more appropriate to use it to estimate the time to corrosion initiation within an interval that reflects the accuracy of the input data.

In addition to models predicting when corrosion will be initiated, models have also been developed to predict when the first damage will occur. These, however, require more input data and were considered inappropriate for the general nature of this research.

The use of any modelling technique for strategic management of a bridge network will involve a bridge-by-bridge evaluation of corrosion risk, similar to the recent seismic evaluation and the scour risk assessment currently underway. This will require the confidence limits to be established for any model used. The model and the confidence limit may vary between prestressed and reinforced concrete. This investigation examined whether a simple model based on Fick’s Law might, in principle, be suitable for prestressed beams but did not seek to establish confidence limits associated with the model.

For prestressed bridge beams such as those examined in this investigation, it is reasonable to assume that the bond may be lost shortly after corrosion initiation. This assumption can be used in a simple strength evaluation model to estimate when each strand/wire can no longer be relied upon in individual elements.

The time to corrosion initiation was calculated from Equation 3 for each element on which chloride profiles were measured. The cover depths used as the critical value were the average cover depths to the stirrups as measured on the webs of the beams from which the samples were taken for chloride analysis. The depth at which the chloride concentration reached the threshold value of 0.05% by weight of concrete was also calculated. The results are in shown Table 6.3. Table 6.3 also presents results based on a chloride threshold of 0.03%.

Table 6.3 Estimates of time to corrosion initiation based on chloride ion

Bridge name (exposure classification)	Span	Beam	Location on web	Average cover to web stirrup (mm)[a]	Depth of threshold chloride content (mm)		Total predicted life (years)[b]		Remaining life as at 2006 (years)[b]	
					0.05%	0.03%	0.05%	0.03%	0.05%	0.03%
Hamanatua (B2)	AB	Upstream	Downstream	28	20	23	80	57	40	16
		Inner	Downstream	35	18	23	150	91	110	50
		Downstream	Downstream	29	21	25	70	53	30	12
	BC	Upstream	Upstream	27	27	31	40	31	0	-10
		Inner	Upstream	32	26	29	60	47	20	6
		Downstream	Downstream	31	24	31	70	38	30	-3
Hawai (B2)	South	Downstream	Downstream	23	35	42	20	11	-20	-27
Kereu (B2)	South	Upstream	Upstream	44[c]	<5	<<5	∞ (∞)	∞ (∞)	∞ (∞)	∞ (∞)
		Downstream	Downstream	39[c]	26	33	100 (54)	57(32)	50 (12)	14 (-11)
Mangahauini No.1 (B2)	Third from north	Downstream	Downstream	32	9	15	∞	195	∞	154
Mangakuri (B1)	South	Inner	Downstream	36	<5	<<5	∞	∞	∞	∞
Oparau (B1)	Northwest	Downstream	Downstream	31	<5	<<5	∞	∞	∞	∞
Waikohu No.3 (A2)	North	Downstream	Upstream	27	<5	<<5	∞	∞	∞	∞

Notes to Table 6.3:

a The average cover depth on each beam was used as the critical value. The critical cover depths used in the calculation have a major effect on the remaining life predictions. If minimum cover depths rather than average cover depths are used as the critical value on each element, the predicted life is much shorter. Similarly, when the average cover depth

6.3.2 Discussion

Although at a very simple level, these predictions support the site observations. They show that corrosion is unlikely on the three bridges in the B1 and A2 exposure zones within the foreseeable future. Of the bridges in the B2 zone, corrosion is also unlikely within the foreseeable future on one element on the Kereu and Mangahauini No.1 Bridges. Corrosion has already been initiated on one element of the Hawai and Hamanatua Bridges, and may be initiated on other elements before these bridges are 100 years old. Corrosion may also be initiated on one element of the Kereu Bridge at around 100 years.

The calculations also show that on many of the beams on bridges in the B2 exposure zone, the depth at which the chloride content had reached the threshold level of 0.05% was close to or less than the specified cover depth of 25 mm. Consequently, corrosion initiation is imminent on many of these beams. On the B1 and A2 bridges, however, the chloride content was this high only very close to the exposed surface.

Closer examination of the data in Table 6.3 reveals some inconsistencies. For example, on the Hamanatua Stream Bridge, the model predicts a 40-year service life at the location where 27 mm minimum cover was recorded. This suggests corrosion of stirrups would be initiated at 40 years, i.e. in 2006. However, extensive corrosion of prestressing strand had already been observed in 2004, suggesting that corrosion was initiated earlier than the model had predicted. The actual cause of the discrepancy was not investigated, but several factors may have contributed.

The most likely factor is the chloride concentration used as the 'threshold' value. In this investigation, 0.05% chloride ion by weight of concrete was used as the critical chloride concentration. A lower value, such as 0.03% (see Chapter 3.3.6), may be more appropriate. Table 6.3 shows that on the basis of a threshold of 0.03%, the corrosion was initiated about ten years ago, which corresponds with observations. This threshold value also suggests that corrosion damage is imminent on one beam of the Kereu Bridge. Further investigation is needed to determine the most appropriate threshold value.

Other factors are as follows:

- **The cover depths measured to the lower corroding strand shown in Figure 5.1 were 55 mm from the beam soffit (design cover is approximately 50 mm) and 35 mm from the side of the beam flange (design cover estimated from the drawing is 43 mm).** These measured covers to the strand are both greater than the 27 mm average cover to the web stirrup upon which the prediction was based. Therefore the discrepancy did not result from overestimation of cover.
- **The apparently premature corrosion observed may be a consequence of an elevated chloride concentration resulting from chloride ingress from both the side and soffit surface.** The flange may be slightly more exposed to chlorides and to rain, and therefore may be more susceptible than the web to chloride ion ingress and faster corrosion rates.
- Or it may have been related to **the sampling technique used in this investigation**. Although appropriate for determining the approximate depth of chloride contamination

and background chloride levels, 20 mm is a relatively coarse depth increment to sample when considering cover depths of 50 mm or less, and smaller increments such as 15 mm or even 10 mm may be more appropriate for chloride ingress data that are to be used for predictive modelling.

Several factors will contribute to inherent inaccuracies in any modelling technique, even when the surface of interest is sampled, the actual cover depths are used and samples taken at small depth increments. These include the variation in surface chloride contents, cover depths and concrete permeability over the length and height of an individual surface, and the relative roles of absorption and diffusion in the ingress of chloride into the concrete. This variability is not practical to quantify for every surface on every structure; therefore the apparent discrepancy between the observed and predicted behaviour may be seen as an indicator of the inherent accuracy of the modelling approach. Nevertheless, both the observations and the model predict a service life shorter than 50 years, which although an imprecise interpretation, is still an important one.

6.3.3 Service life requirements

How do these predictions relate to Transit New Zealand's service life requirements? Transit New Zealand bridges must be designed to achieve a 100-year service life, which is defined as the life beyond which the bridge is expected to become functionally obsolete or uneconomic to maintain in a condition adequate for it to perform its functional requirements. It is equivalent to the NZS 3101/New Zealand Building Code (1992) definition of specified intended life, which requires that no major reconstruction or rehabilitation be required within that period, although routine maintenance may be carried out. The commentary to NZS 3101:2006 points out that normal maintenance may include repair of some surface cracking or minor spalling. It recommends using the Bridge Manual (TNZ 2003) definition of 'major renovation', i.e. maintenance that is necessary to maintain the strength, ductility capacity or serviceability of a bridge to fulfil its functional requirements and that exceeds 20% of the replacement cost of the bridge. This implies that significant damage must not be incurred within 100 years, although minor damage may be acceptable (e.g. isolated spalls over local areas of inadequate cover). Corrosion initiation within 100 years may therefore be acceptable, but corrosion damage that affects serviceability within 100 years would not be acceptable. Because of the inaccuracies inherent in the Fick's Law approach to predicting corrosion activity, in this investigation, a time to corrosion initiation of 100 years is considered, for strategic asset management purposes, to represent a conservative estimate of time to corrosion damage of the rebar and therefore to the end of design life. It is a less conservative estimate of time to corrosion damage when considering prestressing steel because of the increased risks associated with corrosion of prestressing steel.

For conventionally reinforced bridges, the ultimate limit state normally governs structural limitations; whereas on prestressed structures, the serviceability limit state normally governs design. Use of the time to corrosion initiation to define the end of 'service life' for prestressed structures is in accordance with this approach.

Overall, the predictions in Table 6.3 show that if a 100-year service life is to be achieved or exceeded on bridges of this design in the B2 exposure zone, some intervention may be necessary to prevent or delay corrosion damage, but this intervention is unlikely to be needed on bridges in the B1 or A2 exposure zones.

Depending on the level of risk the owner is willing to accept, a different approach may be more appropriate when assessing the needs of an individual structure. For example, although cover depths to the rebar and strands are similar, the consequences of corrosion in the prestressing strands are more significant than the consequences of corrosion in reinforcing bars that has caused a crack or spall. Repair of cracked and spalled concrete is relatively straightforward, and one or even two cycles of this type of repair may be considered as normal maintenance and therefore acceptable. Repair of an element damaged by corrosion of prestressing steel, however, will probably also involve strengthening the element. If the damage is extensive or the bridge capacity already insufficient, the repair may be considered as a major renovation and therefore unacceptable.

6.4 Implications for bridges of more recent design

It had been hoped that the two sets of data collected from the Turihaua Stream Bridge could be used to 'calibrate' the model described in Chapter 6.3 by entering chloride contents from the same depth collected at different times, thereby improving confidence in this modelling approach. The sampling profiles used in 1991 and 2006 were different, and it was felt that the inaccuracies introduced by interpolating chloride contents at given depths would mask any differences related to the 15-year interval between the sampling dates. This method of 'calibration' was therefore omitted from the analysis of data in this investigation.

Instead, Equation 3 was used to predict time to corrosion initiation on the Turihaua Stream Bridge, assuming a range of cover depths and using data collected from the side of the bridge deck in 1991 and in 2006. The results from the 1991 and 2006 sets of data were identical, which gives some confidence in this modelling approach. The results are shown in Table 6.4. The depth at which the chloride content had reached the threshold value of 0.05% was also calculated and found to have increased from 19 mm in 1991 to 28 mm in 2006.

Table 6.4 Estimates of time to corrosion initiation for the Turihaua Stream Bridge.

Cover	Total predicted life (years)*	Remaining life as at 2006 (years)*
15	10	-20
20	10	-20
25	20	-10
30	30	0

* Calculated values rounded to the nearest 10 years.

These results indicate that some intervention may be needed to achieve a 100-year service life on this bridge. No evidence of corrosion was observed on the bridge deck during either the 1991 or the 2006 inspections, so either the predicted time to corrosion initiation is over-conservative or corrosion has occurred without visibly damaging the concrete. Nevertheless, with chloride ion contamination already significant 28 years after construction, some level of corrosion damage is likely within its 100-year service life, and therefore some intervention will be needed. Consideration of the risk to bridges of this design, however, is outside the scope of this investigation.

The bridge beams examined in this investigation were made from 38 MPa concrete and had a specified cover depth of 25 mm. In comparison, NZS 3101: 2006 Clause 3.7 requirements for compressive strength and cover depths for a specified intended life of 100 years in the A2, B1 and B2 exposure zones are shown in Table 6.5. Cover of 25 mm would now be considered clearly inadequate for these exposure zones. The standard bridge beam design (MWD 1978) requires a minimum 28-day compressive strength of 40 MPa, and this is likely to remain unchanged by the revised designs currently being developed (D. Kirkcaldie, pers. comm. 16 March 2007).

Table 6.5 NZS 3101: 2006 Clause 3.7 minimum cover depth requirements for 100-year specified intended life for A2, B1 and B2 exposure zones.

Exposure classification	Specified compressive strength						
	25 MPa	30 MPa	35 MPa	40 MPa	45 MPa	50 MPa	60–100 MPa
A2	50 mm	40 mm	40 mm	35mm	35 mm	35 mm	30 mm
B1	55 mm	50 mm	45 mm	40 mm	40 mm	35 mm	30 mm
B2	–	65 mm	55 mm	50 mm	45 mm	40 mm	35 mm

Based on the performance of 38 MPa concrete observed in the bridges examined in this investigation, will the cover depths of 35, 40 and 50 mm required for 40 MPa concrete be high enough to provide a 100-year specified intended life?

These cover depths were used as critical cover depths in calculations of time to corrosion initiation for the bridges examined. The results are shown in Table 6.6. They support the NZS 3101: 2006 requirements for a 100-year specified intended life, despite the inherent inaccuracies in this approach that are described in Chapter 6.3. The exception may be where an element is extremely exposed, such as the downstream face of the downstream beam on the Hawai Stream Bridge, which is within 100 m of an open surf beach. The NZS 3101: 2006 requirements were based on the same modelling approach and therefore it is not surprising that the results reported herein agree with them. Nevertheless, the NZS 3101: 2006 requirements were developed by pooling data from many structures, many present-day concrete mix designs and results from various predictive models, all based on Fick's Law but each with its own assumptions (Neil Lee, pers. comm. May 2007) so it is encouraging to find that the requirements correspond with observed behaviour on specific structures.

Table 6.6 Service life predictions based on bridges examined assuming cover depths specified by NZS 3101: 2006 for 40 MPa concrete.

Bridge name (exposure classification)	Span	Beam	Location on web	Cover depth (mm)	Total predicted life (years)*
Hamanatua (B2)	AB	Upstream	Downstream	50	∞
		Inner	Downstream	50	∞
		Downstream	Downstream	50	∞
	BC	Upstream	Upstream	50	130
		Inner	Upstream	50	150
		Downstream	Downstream	50	170
Hawai (B2)	South	Downstream	Downstream	50	80
Kereu (B2)	South	Upstream	Upstream	50	∞
		Downstream	Downstream	50	160
Mangahauini No.1 (B2)	Third from north	Downstream	Downstream	50	∞
Mangakuri (B1)	South	Inner	Downstream	40	∞
Oparau (B1)	Northwest	Downstream	Downstream	40	∞
Waikohu No.3 (A2)	North	Downstream	Upstream	35	∞

* 'Life' means time to corrosion initiation. ∞ indicates more than 200 years. Calculated values were rounded to the nearest 10 years.

6.5 Structural implications of prestressing strand corrosion

Having identified that corrosion is likely on a significant number or prestressed bridges before they are 100 years old, the risk associated with the corrosion needs to be evaluated. This was not part of the main aim of the project but is worthy of consideration nonetheless.

Bond failure may be the first mode of failure in prestressed concrete, resulting in the strand or wire slipping in the affected part of the beam. If bond is lost at one or both ends of the beam, the prestressing force applied by the affected strand will be lost. Because of the sudden loss of prestress when the bond is lost, a conservative approach to managing this risk is to assume that the strand/wire is out of service as soon as the criteria for corrosion initiation are met.

In addition, prestressing strands and wires are of much smaller diameter than conventional reinforcing bars, so they lose a larger proportion of their cross-sectional area than reinforcing bars corroding at the same rate. Consequently, for a given corrosion rate, the loss of structural capacity resulting from corrosion of prestressing steel is much more significant than that resulting from corrosion of a reinforcing bar.

Pitting of the steel is likely when the corrosion is associated with the presence of chloride ions, further increasing the likelihood of failure by localised yielding or fracture.

The risk is exacerbated by the presence of other corrosion mechanisms that may reduce the amount of corrosion damage that is needed to cause prestressing steel to fail, e.g. stress corrosion, hydrogen embrittlement/cracking, fretting and fatigue corrosion.

When steel is under tensile stress, corrosion may induce cracking that causes the steel to fail in a brittle mode and at lower stress levels than when unloaded. Some steel compositions are more susceptible to stress corrosion cracking than others (see Chapter 2.2). Stress corrosion cracking can be identified in failed sections by microscopic analysis of the failed section but is difficult to detect prior to failure.

Once the bond has been lost, fretting may reduce the corrosion resistance of the steel.

Pitting and crevice corrosion induced by chloride ions both generate hydrogen ions, which can diffuse into the steel. At low concentrations, this can make the steel brittle; at higher concentrations, it can crack or blister the steel. Both mechanisms cause the steel to fail at lower stress levels. The presence of pitting or crevice corrosion indicates the possibility of these effects, known as hydrogen embrittlement and hydrogen cracking.

The fatigue life of steel is reduced in corrosive environments. The fatigue life of the prestressing steel in these beams, therefore, has probably been reduced by the chloride contamination. Fatigue corrosion is readily identified by microscopic analysis of a failed section, but difficult to detect prior to failure.

Unfortunately, because the strand on the Hamanatua Stream Bridge had not completely failed, it could not be sampled safely. It is therefore impossible to determine from the work carried out to date whether the steel is of the type specified and whether it is at significant risk from stress corrosion, hydrogen embrittlement/cracking or corrosion fatigue. Even if the risk from any one of these mechanisms is slight, the combination may increase the overall risk.

Darmawan & Stewart (2007) reported that cold-drawn, stress-relieved prestressing wires and strands (manufactured in accordance with AS 1310-1987 (Standards Australia 1987a) and AS 1311-1987 (Standards Australia 1987b) respectively) had a lower strain at failure when corroded than when not corroded. They found no evidence of stress corrosion cracking or brittle failure, but considered that although the steel yielded, its failure was less ductile than on uncorroded companion specimens. They suggested that the failure mode in any particular case was related to the pit geometry, stress level and chemical environment as well as to the properties of the steel itself. Failure modes may therefore be determined by the specific materials and conditions on individual structures.

Further work will be needed to identify the most likely strand failure mechanisms in the prestressing steel used in the beams investigated here. As noted in Chapter 6.1, however, such a study is not essential for maintaining the serviceability of the bridges.

The consequence of any failure mechanism is a reduced capacity of the beam to withstand design loads. No measurements were taken on site to assess the residual prestress and thereby to assess the serviceability of the structure. Nevertheless, on the basis of the calculated effect of removing the two affected strands, the loss of beam and superstructure capacity caused by the corrosion observed at the Hamanatua Stream Bridge was assessed to be up to 10% loss in live load capacity in the individual beam which has lost the two strands. In this condition, the overall reduction in the live load capacity of the superstructure is estimated to be up to 5%. This reduction in capacity from the original design becomes even more significant when considering that the bridge was designed to a lower standard than currently required by Transit New Zealand.

Because corrosion of as few as two strands can significantly reduce the load-carrying capacity of bridges of this design, it is important that further work be carried out to understand and quantify the risks involved.

6.6 Long-term management of pre-1973 prestressed bridges

This investigation has identified that reinforcing and prestressing steel in bridges of this particular design in the B2 (or C) exposures zones is likely to corrode within 100 years from the time of construction. The associated structural risk is significant because the strand most likely to be affected without prior warning is not confined, and because the design load capacity of these structures is already lower than current Transit New Zealand requirements.

Further investigation is needed to ascertain the impact of prestressing strand corrosion on individual bridges such as the Hamanatua Stream Bridge to enable suitable remedial strategies to be developed. This includes determining the likely strand failure mechanism and the cross-section loss that corresponds to bond loss, and developing a site sampling regime that will allow chloride-induced corrosion on individual structures to be modelled accurately by techniques based on Fick's Law.

Irrespective of the structural risk, intervention to prevent or delay the onset of corrosion will be necessary on some bridges. Such intervention may be preventive, such as applying a water-resistant surface treatment to the concrete surface before chloride contamination has reached critical levels. Preventive action should be taken before the predicted time to corrosion initiation. Alternatively, intervention may mean repairing damaged elements once the concrete has cracked or spalled. Further research is needed to assess the economic benefits of these two approaches so that the most appropriate option can be selected, either for individual bridges or for bridge populations.

In the meantime, as the first stage of developing a strategy for managing these bridges, the risk to prestressed bridges on the roading network needs to be identified. Because the beam design means visible signs of corrosion are often absent before failure, non-destructive methods of detecting corrosion activity such as those described by Ali & Maddocks (2003) would be useful, although, as noted in Chapter 2.5, a combination of techniques is required in order to be effective. A more practical approach based on a simple site investigation of each of the bridges is therefore recommended. The cover depths and the level of chloride ion contamination should be measured on all pre-1973 prestressed I-beam bridges in the B2 exposure zone (approximately within 1 km of the coast) in New Zealand. The bridges should be inspected to identify corroding prestressing and reinforcing steel, and routine inspection reports should be reviewed to find out when any such corrosion damage first occurred. This will enable bridge owners and asset managers to determine the potential scope of the problem and to budget for appropriate remedial or preventive action to ensure that these bridges continue to perform satisfactorily for the desired service life.

Further work as described above would allow the loss in capacity over a given period (e.g. 10, 20 or 30 years) to be estimated. This would enable asset managers to estimate the risks and benefits associated with carrying out remedial work at a given time and thus optimise the time at which such work is done.

Application of an appropriate surface treatment before the steel corrodes will prevent or delay corrosion damage. Several proprietary surface treatments for this purpose are available. Further work is required to compare the advantages and limitations offered by the different products.

Options for repairing concrete damaged by reinforcement corrosion are described by Freitag & Bruce (2002) and Freitag et al. (2003). Further work is needed to establish the effectiveness of these techniques for repairing damage caused by corroding prestressing steel. For example, repairing spalled or cracked areas will not reinstate bond that has been lost elsewhere on the beam without visibly damaging the cover concrete. It may also be useful to assess the potential benefits of preloading a beam during repair to put the exposed strand into tension then releasing the load to put the completed repair into compression.

Treatments available to restore and improve structural capacity include external post-tensioning and retrofitting with fibre-reinforced polymer composites. Proprietary composite systems suitable for bridge applications are available. Further research is needed to identify the benefits, limitations and constraints on the use of each of these options for strengthening typical New Zealand concrete bridges damaged by corroding prestressing steel.

These bridges were built to standards that do not meet the requirements of the current Transit New Zealand Bridge Manual (TNZ 2003), even though they may be performing acceptably. Should major structural improvements be needed, the rehabilitation strategy for each bridge will need to account for the design life of the upgraded structure. Therefore, future research into strengthening options also needs to consider the future durability of the structure.

7. Conclusions

- **Cause and extent of prestressing strand corrosion on the Hamanatua Stream Bridge (built in 1966):**
  - The bridge is in the B2 'coastal frontage' exposure zone and within 200 m of an open surf beach.
  - Prestressing steel on at least one beam is corroding and has spalled the cover concrete, but the strand has not yet broken.
  - Concrete quality and workmanship is generally good.
  - Cover to some of the steel was less than the minimum specified cover of one inch (25 mm).
  - The corrosion was caused by the ingress of chloride ions from sea spray, resulting in chloride ion concentrations at the steel surface exceeding the threshold value at which corrosion is initiated.
  - The influence of prestressing steel composition and different corrosion mechanisms on the observed deterioration could not be determined.
- **Corrosion observed on other bridges of this design and age:**
  - Prestressing strand corrosion was not observed on any other bridges of this design.
  - Corrosion of conventional reinforcement was relatively common.
  - In some cases, the small volume of corrosion product has not yet generated sufficient stress to damage the cover concrete.
- **Likelihood of future prestressing strand corrosion on bridges of similar design and age to the Hamanatua Stream Bridge:**
  - *In situ* and laboratory testing showed that the same corrosion mechanism is likely to affect bridges in the B2 exposure zone because the amount of chloride contamination, the quality of concrete and the depth of cover are similar to those on the Hamanatua Stream Bridge.
  - Corrosion is unlikely in bridges in the B1 and A2 exposure zones because they are not exposed to external sources of chloride ion contamination.
  - Analysis of concrete samples from these bridges revealed that the concrete in some of them contains calcium chloride accelerating admixture, which increases the likelihood of corrosion irrespective of exposure conditions.
- **Predicted times to corrosion (service life) on bridges of this design and age:**
  - The approximate time to corrosion initiation can be predicted by a simple model based on chloride ion diffusion rates. Bond between the concrete and prestressing steel may be lost shortly after corrosion initiation, so time to corrosion initiation is a reasonable approximation of time to corrosion damage;
  - The model predicts that bridges in the B2 exposure zone are unlikely to achieve a 100-year service life without some corrosion damage.
  - The model predicts that bridges in the B1 and A2 exposure zones will probably achieve a 100-year service life with no corrosion damage.
  - The model's predictions broadly correlate with observations.

   - Refinements to the sampling and modelling procedures may allow more precise predictions.
- **Likelihood of corrosion on bridges of more recent design:**
   - Beams designed to current specifications (NZS 3101: 2006 and Transit New Zealand's Bridge Manual (TNZ 2003)) have much greater concrete cover depths than the pre-1973 bridges investigated during this research.
   - The corrosion initiation model indicated that beams made with similar concrete quality to the Hamanatua Stream Bridge but with cover depths in accordance with current specifications will probably achieve a 100-year service life without corrosion damage.
   - Beams with similar concrete properties and cover depths to the Hamanatua Stream Bridge do not comply with current specifications and are just as likely to be affected by corrosion.
- **Structural implications of prestressing strand corrosion:**
   - For a given corrosion rate, corrosion in the prestressing strand reduces the structural performance of a beam faster than corrosion of conventional reinforcing because a greater proportion of the steel cross-section is lost.
   - The Hamanatua Stream Bridge beam on which corrosion was observed may have lost up to 10% of its live load capacity, reducing the overall load capacity of the superstructure by up to 5%.
   - The possible influences of hydrogen embrittlement, stress corrosion cracking, fretting corrosion and fatigue corrosion on the eventual strand failure mode on the Hamanatua Stream Bridge could not be determined because the strand could not be sampled safely for investigation.
- **Implications for long-term management of pre-1973 prestressed bridges:**
   - The risk associated with prestressing steel corrosion in these bridges is greater than the risk of reinforcing corrosion in bridges of similar age.
   - Some intervention will be necessary to ensure that bridges of this particular design in the B2 exposure zone remain serviceable for a 100-year service life.
   - Intervention may involve either preventive maintenance or repair to concrete once the steel has started to corrode.
   - Further work is needed to determine the most effective preventive and remedial techniques, and the most economic strategy.

8. Recommendations

8.1 Findings from current research

Transit New Zealand and other bridge controlling authorities should consider the findings of the research reported herein when assessing the risks to their bridges as part of their asset management programmes.

8.2 Further work

To enable Transit New Zealand and LTNZ to manage the corrosion risk identified by this project in pre-1973 prestressed concrete bridges (and in more recent designs) cost-effectively and proactively, further work is recommended to identify the specific bridges at risk in the B2 exposure zone and to identify appropriate methods of managing prestressing corrosion in these bridges. Appendix A indicates what this work should entail.

Once the bridges at risk in the B2 exposure zone have been identified and appropriate methods of managing prestressing steel corrosion have been identified, the incidence and severity of this type of corrosion in B1 and A2 exposure zones should be investigated. This work may not be necessary for another 10–20 years.

To refine the methods of predicting corrosion initiation and to optimise mitigation strategies, further research may be carried out to improve the precision and accuracy of corrosion predictions, and ascertain the structural impact of prestressing strand corrosion on individual bridges. Appendix A suggests topics to be included in such research.

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Appendix A Details of proposed further work

A1 Outline

Chapter 8 of this report presents general recommendations from the findings of this project. These include carrying out further work to facilitate proactive management of pre-1973 prestressed concrete bridges, and further research to refine the methods of predicting corrosion initiation and to optimise mitigation strategies. The suggested approaches to the recommended work and research are described in this Appendix.

A2 Recommended further work

To enable Transit New Zealand and LTNZ to manage the corrosion risk identified by this project in pre-1973 prestressed concrete bridges (and in more recent designs) cost-effectively and proactively, the following work is suggested:

- **Identify the bridges at risk by:**
  - identifying by review of construction records whether the prestressing steel used in the Hamanatua Stream Bridge had a particularly low corrosion resistance and whether the same prestressing steel was used in all bridges of this design;
  - identifying the precast concrete manufacturer(s) that made the beams believed to contain calcium chloride accelerating admixture, and identifying other bridges that have beams from the same source;
  - finding out how the overall risk to prestressed concrete bridges of other designs compares to the risk to the pre-1973 bridges by examining critical design features such as cover depths and confinement of prestressing wire/strand, and ascertaining the age and geographical distributions of each design;
  - developing a site testing and evaluation procedure that would allow the corrosion risk on individual bridges to be assessed with a quantified accuracy; and
  - using this procedure to assess the likelihood of current and future corrosion on all pre-1973 prestressed I-beam bridges in the B2 exposure zone (approximately within 1 km of the coast) in New Zealand. The bridges should be inspected to identify corroding prestressing and reinforcing steel, their routine inspection reports reviewed to find out when any such corrosion damage first occurred, cover depths and levels of chloride ion contamination on the beams measured and the time to corrosion initiation should be calculated via the simple model used in this project. This will enable bridge owners and asset managers to determine the potential extent of the problem and to budget for appropriate remedial or preventive action to ensure that these bridges continue to perform satisfactorily for the desired service life.

- **Identify appropriate methods of managing prestressing corrosion in these bridges by**:
  - identifying the benefits and limitations offered by various surface treatments to inhibit the ingress of moisture and chloride ions into the concrete surface[1];
  - assessing the benefits and limitations of patch repair techniques and cathodic protection for repairing damage caused by corrosion in prestressing steel;
  - comparing the economic benefits of preventive maintenance and remedial techniques so that the most appropriate option can be selected, either for individual bridges or for bridge populations, taking the condition and structural capacity of the bridges and their environmental exposure conditions into account;
  - identifying the benefits, limitations and constraints on the use of methods such as post-tensioning and fibre reinforced composites for strengthening typical New Zealand concrete bridges damaged by corrosion in prestressing steel, including their impact on durability and the effect of further deterioration on their integrity.
- **Once the bridges at risk in the B2 exposure zone have been identified and appropriate methods of managing prestressing steel corrosion have been identified, investigate the incidence and severity of this type of corrosion in B1 and A2 exposure zones in more detail.** This work may not be necessary for another 10–20 years.

A3 Further research

To refine the methods of predicting corrosion initiation and to optimise mitigation strategies, the following research is suggested:

- **improve the precision and accuracy of corrosion predictions by:**
  - correlating observations of prestressing condition with measured chloride contamination to ascertain the most appropriate level of chloride contamination to use as a 'threshold' value when predicting time to corrosion initiation; and
  - determining the likely *in situ* range of concrete properties and other input data used to model and predict the corrosion initiation from a given element.
- **ascertain the structural impact of prestressing strand corrosion on individual bridges such as the Hamanatua Stream Bridge by:**
  - determining by laboratory analysis of samples from failed prestressing steel (should any be encountered) whether the eventual failure mechanism of the prestressing steel in the pre-1973 bridges is likely to include stress corrosion, hydrogen embrittlement, fretting and/or corrosion fatigue; and
  - ascertaining whether the strand is likely to yield or undergo brittle failure.

[1] LTNZ funded research on this topic during 2007/8.

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