

To: DataTakeON@waipadc.govt.nz
Subject: mail3624624 ECM Data TakeON
Attachments: mail3624624_att1_image001.png, mail3624624_att2_Victoria Bridge (Cambridge) - Resilience Review Report - Cover Letter.pdf, mail3624624_att3_Victoria Bridge (Cambridge) - Resilience Reivew Report.pdf

Message: 18091519

From: [Mekhala Kariyawasam](#)
To: [Bryan Hudson](#)
Cc: [Michael Kulpa](#), [Will Pank](#), [Andrew Ball](#)
Sent: 14/8/2018 at 5:14 PM
Received: 14/8/2018 at 5:15 PM
Subject: Waipa DC - Victoria Bridge (Cambridge) Resilience Review Report

Attachments: [image001.png \(59 KB\)](#)
[Victoria Bridge \(Cambridge\) - Resilience Review Report - Cover Letter.pdf \(197 KB\)](#)
[Victoria Bridge \(Cambridge\) - Resilience Reivew Report.pdf \(1 MB\)](#)

Hi Bryan,

Please find attached the cover letter and the resilience review report prepared for Victoria Bridge. Note that the attached report contains the scope of services specified under Stage 1 in our submitted offer of service (Offer of Service No. 17).

I understand you need to undertake a review prior to proceeding to the next stage. Please do not hesitate to contact us if you would like to discuss anything regarding the Stage 1 works undertaken. I am also available to go through the report findings and have a quick discussion with you after the monthly meeting tomorrow.

Regards,

Mekhala Kariyawasam
Structural Engineer
Beca
Phone +64 9 300 9000 Fax +64 9 300 9300

Mobile +64 27 404 9798

HYPERLINK "http://www.beca.com/"www.beca.com

HYPERLINK "http://www.linkedin.com/company/Beca"www.Linkedin.com/company/Beca

HYPERLINK "mailto:mekhala.kariyawasam@beca.com"mekhala.kariyawasam@beca.com

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Waipa District Council
Private Bag 2402
Te Awamutu 3840
New Zealand

14 August 2018

Attention: Bryan Hudson

Dear Bryan

Contract No. 28-17-03: Hamilton City, Waipa & Waikato District Councils: Bridge & Structures Professional Services

Issue Letter - Victoria Bridge (Cambridge) - Resilience Review Report

Further to our recent discussions and offer of service approved on 23 May 2018, we are pleased to attach Victoria Bridge (Cambridge) – Resilience Review Report as part of our Stage 1 deliverables for comment.

There are a number of unknowns due to missing information with regards to the history of the bridge, which we hope your records may be able to clarify. Should you be able to provide clarity on these we can incorporate these into the report as part of any amendments that may come from your comments. The missing information is as follows:

- It is not clear from the information currently available to us whether the repainting and repair works recommended in the 1996 report were carried out. It is thought that at least some of the work may have been carried out in 1999 however the extents of this work need clarifying.
- It is not clear from the information currently available to us when the last bridge repainting was carried out. It is known that the bridge was repainted in 1978 with the inclusion of a red lead primer and it is thought that this may have been replaced as part of the 1999 works above from comments in the 2011 inspection report. Previous discussions with the council have implied that there may have been subsequent repainting since this time.
- We do not have information on the remediation works completed in 2012 on the slope below the southern arch foundations. This will be needed for the slope assessments in Stage 2.
- As part of the design of the footpath widening works in 2012 an assessment was carried out and it was recommended to install shear connectors through the top flange of the back-span girders. The as-builts of the footpath works do not include this work. There is no evidence of this having been carried out or of further analysis eliminating such a requirement.

Following closure of the above we shall look to confirm with you the scope and costs for Stage 2.

Please do not hesitate should you wish to discuss any of the above.

Yours sincerely



Mekhala Kariyawasam

Structural Engineer

on behalf of

Beca Limited

Direct Dial: +64 27 404 9798

Email: mekhala.kariyawasam@beca.com

Yours sincerely



Will Pank

Technical Director - Structural Engineering

on behalf of

Beca Ltd

Direct Dial: +64 9 308 4880

Email: will.pank@beca.com

Report

Victoria Bridge (Cambridge) - Resilience Review Report

Prepared for Waipa District Council

Prepared by Beca Limited

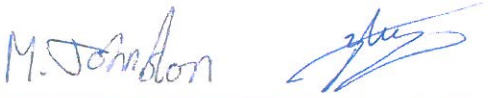
14 August 2018



Revision History

Revision N°	Prepared By	Description	Date
1	Mike Johnston	Issue for Information	14 August 2018

Document Acceptance

Action	Name	Signed	Date
Prepared by	Mike Johnston / Zac Lyon		14/08/2018
Reviewed by	Will Pank / Nathan McKenzie		14/08/2018
Approved by	Will Pank		14/08/2018
on behalf of	Beca Limited		

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Executive Summary

Background:

Victoria Street Bridge (RP 969) also known as Cambridge High Level Bridge is located in Cambridge within the Waipa District, Waikato Region. The bridge is of great local importance and is a Grade 1 listed structure by Heritage New Zealand.

There are two vehicle routes in Cambridge between the northern and southern sides of the Waikato River that bisects the town. A traffic capacity and demand study dated January 2018 concluded that there was no immediate need for additional crossings provided that the current crossings remain in service. However the report highlighted the risk posed to the traffic flows should Victoria Bridge be taken out of service and recommended that an investigation on the structures likely lifespan be undertaken.

This report intends to determine what level of investigations and planning is required for the Council to have a level of confidence in the long term resilience of the structure.

Findings:

Victoria Bridge is generally in good to fair condition based on the current inspection information.

The key structural defects are in keeping with the bridge's age and include small pockets of corrosion and localised section loss, cracking in the concrete deck and deteriorating paintwork.

The primary issue for determining future service life and maintenance costs lies in the lack of a recent Principal Inspection and associated corrosion maintenance plan. Also the condition of the deck, flagged as a concern in the 2014 inspection, needs to be investigated.

Slope instability is a potential vulnerability for the structure that has not been assessed to date due to lack of information on the bridge founding substrata. Geotechnical investigation work followed by an assessment of the slopes in the vicinity of the bridge footings is recommended.

Combining the knowledge from the assessment and investigation work a plan detailing any immediate repairs or strengthening work can be developed alongside a future maintenance plan. For efficiency any maintenance works should be carried out alongside the repainting works already planned to take place.

While the bridge has a vehicle weight limit and a speed limit in place the risks to the public crossing the bridge are generally assessed to be low.

Recommendations:

A staged approach to the proposed resilience review is recommended as follows:

Stage 1. Detailed Principal Inspection, geotechnical and defect investigations.

Stage 2. Detail current repair and repainting works and prepare long term maintenance strategy.

Stage 3. Carry out the repairs and repainting work.

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Technical Geotechnical Review

1 Introduction

Victoria Street Bridge (RP 969) also known as Cambridge High Level Bridge is located in Cambridge within the Waipa District, Waikato Region.

The Waikato River bisects Cambridge, separating the town centre and surrounding residential areas on the northern side of the river from the residential areas of Leamington and surrounds on the southern side of the river.

There are two vehicle routes between the northern and southern sides of the river within Cambridge, Victoria Street and Shakespeare Street. A traffic capacity and demand study entitled “Cambridge Town Centre Road Bridges Capacity and Demand Study” was produced by Beca Ltd (Beca) for Waipa District Council (Waipa DC) in January 2018. This report concluded that there was no immediate need for additional capacity on the network provided that the current crossings remain in service. However, the report highlighted the risk posed to the traffic flows should Victoria Bridge be taken out of service and recommended that an investigation on the structure’s likely lifespan be undertaken.

Subsequently Waipa DC have engaged Beca to carry out a study on the structural and geotechnical resilience of Victoria Bridge. The scope of which is as follows:

- A desktop study to provide an overview of potential structural and geotechnical vulnerabilities of the structure.
- A high level review of the previous structure and associated live load assessment.
- A review of the available geotechnical information.
- A geotechnical inspection of the bridge and surrounding slopes.
- Confirmation of assessment parameters for any future ground analysis.
- Identify potential risks to the public.

The intent of this initial study is to determine what level of investigations and planning is required for the Council to have a level of confidence in the long term resilience of the structure. It should be noted that the current scope does not include a structural inspection or re-assessment of the bridge, the desk study is summarised in this report and detailed along with the sources of information in the appendices.

2 History and Structural Form

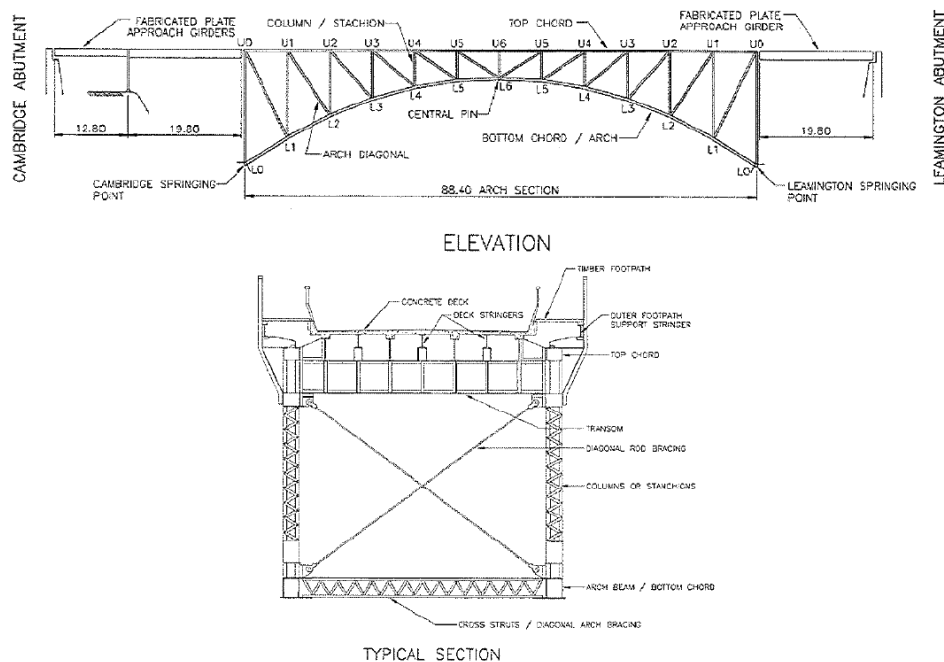
Opened in December 1907 Victoria Bridge is 111 years old. It is the first example of the rare braced arch bridge in New Zealand and has been the main link across the river for most of its history. The bridge is of great local importance and is a Grade 1 listed structure by Heritage New Zealand, therefore demolishing or significantly altering the structure is not permitted and any modifying works need to be in keeping with its heritage significance.

The bridge comprises a four span steel structure with a reinforced concrete in-situ concrete deck. The 88.4m long main arch span over the Waikato River comprises a pair of three-pinned braced arch trusses, one on each side of the bridge. The trusses are connected to one another along the arch ring and at each of the vertical members by cross bracing. The flanking spans are 19.8m in length and formed of plate girders. An additional 12.8m plate girder span is present at the Cambridge end of the bridge.

The arch springs from pin connections on the concrete piers 15.2 m above water level with a rise of 15.8 m to the crown pins at the centre. The arch crown and springing pins lie in an arch of 69.5 m radius. This arch extends the full width of the river and is divided into twelve panels by vertical columns and diagonal stanchions. The arch span is considered to act as a truss. Arch members are formed from riveted laced battens with the smaller members fabricated from channels and plates and the larger ones fabricated from steel plates and angles. Transom members span between the heads of the columns and diagonal rod bracing provides lateral restraint. An elevation and cross section of the structure is shown in Figure 2-1.

The deck is supported on the transoms with four longitudinal girders and three intermediate stringers. The stringers are supported at intervals by crossbeams between the main girders. The original deck was timber and has since been replaced by an approx. 95mm thick concrete deck slab with cantilevered footpaths added on either side. Multiple services are located beneath the deck.

Figure 2-1 Typical elevation and section of the bridge (through arch).



2.1 Timeline of Modifications

The desk study has identified the following key modifications to the existing structure:

1907: Bridge opened based on a design from 1905.

1925: Clip on footpaths added to either side of the bridge following a design completed in 1919 by the Public Work Department. Softwood deck replaced with hardwood deck.

1950: Extensive repairs, any section with corrosion over 2mm in depth replaced. Compression spreaders added across the bottom of the arch. Steel replacement along the bottom of the arch web by welding 10mm flat bar into the intersection of the web plate and angle leg. First welding on the bridge thought to have taken place. 9" x 3" running planks replaced by 9 x 2" planks which reportedly reduced the dead load by 86.8 ton.

1964: Timber deck replaced with the current concrete deck. Extensive sandblasting and repainting also undertaken, original handrail removed from its location between road and footpath and replaced with pipe rail that is currently in place. New 250mm diameter water main replaced two old 175mm pipes. Footpath timbers replaced.

1976: Felspan expansion joints installed.

1978: Major repairs carried out replacing all of the original diagonal bracing in the plane of the arch with new members constructed of rectangular hollow sections fabricated to retain the diagonal lacing effect. The diagonal bracings over the springing points were also replaced with these RHS fabricated members.

Other repairs included rivet, lacing and rod bracing replacements as well as section build up in areas of high corrosion. The deck was also resealed and the outer handrail replaced.

The bridge was sandblasted and repainted with a red lead primer and top coated with two layers of micaceous iron oxide paint. This was the last time the bridge was fully repainted.

1982: Gantry rails added beneath outer handrails.

1984: Diagonal braces between truss nodes L5 and U5 replaced with RHS fabricated members. Overhead height limitation signs and gantries introduced due to the height and weight limitations being consistently ignored.

1991: Drainage extensions added to existing drainage system to avoid drainage out falling onto structure below.

1999: It is thought that repairs and repainting were completed following the 1996 inspection and assessment. No documentation for this has been found however 2011 inspection refers to painting at this time.

2012: Following identification of slip defects to the south abutment slope in a 2011 inspection report soil nailed slope stabilisation works beneath pier D footings were installed along with erosion protection mesh and matting as well as slope drainage and planting.

2014: Cantilevered footpaths replaced to provide additional width for pedestrians and cyclists. Plain RHS sections used for majority of components, Permaslab Fibre Reinforced Polymer (FRP) used as decking to reduce additional dead load. New height restriction gantries installed. Steel angles bolted to the top flanges of longitudinal beams to repair corroded element. Repairs to top chord lacing bars.

3 Condition

The main defects that steel bridges commonly present are corrosion and member section loss. With any steel bridge corrosion pockets are expected, their extent and severity are determined by inspection and the condition translated into a reduction in calculated capacity through use of a condition factor. The condition of Victoria Bridge is monitored during regular inspections where defects are identified for repair or maintenance.

These defects are generally prevented by use of paint systems, drainage systems and removal of vegetation. Critical defects are remediated by removal of the corroded areas and either replacing the steelwork or adding new steelwork adjacent to the affected areas.

Victoria Bridge is covered as part of the regional bridge inspection programme. There are different levels of inspection that can be carried out for a structure varying from a general visual inspection, known as a General Inspection, to a full detailed inspection, known as a Principal Inspection. The difficulty in gaining access to all areas of Victoria Bridge limits the amount of inspection that can easily be carried out on a regular basis. For this reason general inspections have been regularly carried out while special inspections, using specialist access equipment, have been relied on in place of regular full inspections.

After reviewing the existing inspections of the bridge it can be summarised as being in good to fair condition.

Defects of note include:

- The expected corrosion pockets which tend to be occurring in the sheltered areas of the bridge such as at the connection points between truss elements and immediately below the bridge deck.
- Deformed lattice bars within truss elements.
- Corrosion at the central pins restricting the original intended movement of the structure.
- General paint degradation in line with the age of the current paint system.
- Cracking and honeycombing of concrete deck including corrosion of reinforcing bars.

Recent inspections have identified the following key maintenance actions:

- Remove and repair areas of corrosion.
- Repaint the structure to prevent future corrosion.
- Carry out concrete repairs and treat exposed reinforcement.
- Install bird protection to reduce the corrosion caused by nesting birds.

4 Assessments and Restrictions

Structural assessments are carried out to determine the compliance of the structure with appropriate standards and to check the capacity of the bridge to resist various types of load effects. Key load cases include live load on the bridge, wind and earthquake events. The levels of earthquake and wind loading that the structure must be able to withstand are determined nationally while the assessment traffic loading determines the restrictions imposed on the bridge.

Traffic restrictions on bridges are known as postings, the posting of bridges is reviewed on an annual basis by the incumbent engineering consultancy. These postings are updated if significant deterioration or strengthening works have occurred since the last assessment of the structure.

An axle weight restriction has been in place since at least 1950 where a 5 ton limit was imposed alongside other restrictions on logging trucks and stock loading. When the concrete deck was introduced in 1964 the restriction was further reduced to 2 tonnes and a 25km/h speed limit imposed. Gantries were introduced to limit the height and width of traffic to further discourage use of the bridge by large vehicles.

The last full assessment of the bridge was carried out as part of a report entitled “Victoria Bridge – Cambridge, Structural and Associated Traffic Analysis” dated September 1996 by BBO. The report checked traffic loading, dead load, wind load and seismic load cases. The report concluded that the bridge could not sustain the standard traffic loading and that the existing 1964 limits were still valid. The limiting component was found to be the deck slab.

Following an increase in the average weight of family vehicles (due to the prevalence of SUVs) this limit was reviewed in 2009 and the limit raised to 3 tonnes so as not to restrict common daily traffic, the speed was also raised to 30km/h. The 2009 review did not include a full assessment of the structure but only analysed the concrete deck as this was the limiting component identified in the 1996 assessment. The new live load effects were compared against the capacity of the steelwork calculated in 1996 and found to comply with standards.

In 2012 design work was undertaken for the increase in footpath width on the structure. During this process the steelwork was also assessed for traffic loading and found to comply with assessment standards with the load limit in place. The concrete deck was not checked at this stage as it was not relevant to the footpath widening design.

The outcome of the assessments described above results in the current posting:

3000kg gross weight, 30km/hr speed restriction, 3m height, 2.1m width.

5 Potential Risks

The key risks to users of the bridge other than those associated with general structural and geotechnical risks are as follows:

■ Pedestrians falling from height

The 2014 installation of a new footway included a new pedestrian parapet. This parapet was designed to the current safety standards with a parapet height of 1.4m to account for risks associated with cyclists using the footway. The risk of accidentally falling from height is therefore controlled by compliance with standards.

■ Errant vehicles

A vehicle containment system is not present on the bridge, and neither the kerb height nor the handrail are sufficient to contain a vehicle to the current design standards. When the 2014 works were carried out a departure from standards was requested to avoid the need for a new vehicle barrier to be installed and also to avoid the need for designing the footpath to take the load of an errant vehicle. The reasoning behind this departure from standards was that for a compliant system to be installed the whole deck would likely need to be replaced and the superstructure would require major strengthening.

Considering the restrictions imposed on the bridge traffic; its limited deck capacity and heritage status, this appears to be a reasonable approach to take as the risk can be considered low of such an event occurring. The bridge is straight over its span and has narrow lanes which means glancing blows to the rail and/or kerb are the only likely impacts.

■ Traffic collisions

The lane width is narrow at 2435mm compared to the desirable 3500mm width generally allowed for in new bridges which may increase the likelihood of vehicle collisions. Due to the restrictions on the bridge any collision is likely to be low speed and therefore of low severity. To reduce the risk of low speed collisions flexible post bollards have been installed along the centreline of the bridge to guide traffic. However, it is considered that such collisions may occur despite the lane guidance.

Provision of a wider lane would reduce this risk. In order to provide a wider lane width, either the bridge deck would need to be widened or the number of lanes reduced. It is not considered feasible for the structure to support the additional dead load and unrestricted traffic caused by an increased deck width without an extensive and likely cost prohibitive strengthening programme. For this reason the deck widening option has been discounted on numerous occasions in the past. The option to reduce the bridge to a single lane controlled by traffic lights has been proposed before, however, it is considered that the impacts to the traffic flows and local community make this less desirable than the current situation.

6 Discussion

Victoria Bridge is a key route both for vehicle and pedestrian traffic as well as critical services connecting the two parts of Cambridge divided by the Waikato River. In order to determine a long term strategy for the town's infrastructure it is important to know how vulnerable this key route is and what expectations can be made of the bridge's service life and upcoming maintenance requirements.

6.1 Structural Vulnerabilities

Identified structural vulnerabilities are set out briefly below. Refer to Appendix A for a more detailed review.

The last detailed structural analysis of the whole structure was carried out in 1996 when the remaining service life of the structure was estimated to be in excess of 30 years. From the available inspections the bridge is considered to be in good to fair condition with defects in line with the age of the bridge.

As is common among steel structures the key issues of structural integrity for the superstructure are associated with corrosion and section loss of elements. It is known that Victoria Bridge does have corrosion present, some severe, particularly around nodes and the central pins.

The structure does not have a full Principal Inspection on record and the majority of the bridge has not been inspected in detail for many years with a reliance on the limited special inspections to flag any issues with its condition or defects. The lack of a substantial condition survey and associated maintenance strategy raises the risk both to the structure's integrity and to unforeseen maintenance costs. As the assessment relies on accurate knowledge of the condition, the outcomes of an inspection may have an impact on the assessed capacity of the bridge requiring repair and strengthening work to be undertaken.

While the current seismic analysis is considered to be valid the 2012 calculations note that, should the structure be considered to provide a lifeline route following a major seismic event, the Council may wish to carry out a more detailed analysis. However, as there is an alternative route via State Highway 1 the Victoria Street crossing is not considered a lifeline route.

6.1.1 Deck capacity risk

The 95mm depth of the concrete deck slab governs the capacity of the bridge and means that the integrity of the deck is particularly susceptible to any defects that may occur.

The honeycombing, cracking and reinforcement corrosion observed in the concrete deck are highlighted as a potential risk and need to be investigated further. The concrete investigations should include the removal of asbestos sheets over a land span to allow for future observation as well as crack mapping and concrete coring to the deck to determine strength and to examine for evidence of defects.

6.2 Geotechnical Vulnerabilities

Identified geotechnical vulnerabilities are set out briefly below. Refer to Appendix B for a more detailed review.

6.2.1 Earthquake Risk

The bridge was constructed at a time when earthquake loading was not well understood and was not a design requirement. Potential earthquake effects could include instability of the river bank slopes supporting the bridge foundations. The vulnerability of the bridge to damage associated with earthquake induced slope instability of the river banks is not known.

Geotechnical investigation and design checks are recommended to be completed to assess the effect of a design earthquake event on river bank slopes supporting the bridge.

6.2.2 Liquefaction

The natural materials within the river banks supporting the bridge have the potential to liquefy in a moderate to large earthquake event if saturated. Liquefaction effects include a significant strength loss that could make any slope instability in an earthquake larger and with a greater impact on the bridge.

6.2.3 Slope Instability

The river bank slopes are steep and appear subject to periodic instability. Instability may be triggered by elevated water levels (eg after heavy or prolonged rainfall), erosion or potentially earthquakes.

Stability losses to the slopes may undermine the soils supporting the bridge foundations, reducing their load carrying capacity.

6.2.4 River Erosion

The Waikato River is subject to bed degradation, resulting in the river bed dropping over time and steepening of the lower river banks. The shallow founded bridge foundations are supported by the lower river bank slopes. Ongoing erosion over coming years to decades may result in instability of the lower river banks, reducing support to these abutment foundations.

6.2.5 Surface Erosion

Sandy soils were exposed in the upper river bank slopes. Localised surficial erosion was noted at the locations of what appear to be stormwater drainage for the bridge approaches. These areas may be vulnerable to ongoing erosion losses, potentially occurring during heavy rainfall events that generate high stormwater flows. Concentrated discharges of stormwater can also trigger slope instability by raised water levels locally.

7 Conclusions and Recommendations

Victoria Bridge is generally in good to fair condition based on the current inspection information. A number of risks have been highlighted in this study and mitigation measures proposed to address them as summarised below.

- The primary issue for determining future service life and maintenance costs lies in the lack of a recent Principal Inspection and associated corrosion maintenance plan.
- In order to provide a level of confidence in the medium and long term resilience of the structure a full inspection of the structure should be undertaken. Also the condition of the deck, flagged as a concern in the 2014 inspection, needs to be investigated including localised asbestos sheet removal and concrete testing.
- A geotechnical assessment and associated ground investigations is required to determine the level of risk associated with instability of the founding materials.

Combining the knowledge from the inspection and investigation work a plan detailing any immediate repair or strengthening work can be developed alongside a future maintenance plan. For efficiency any repair works should be carried out alongside the repainting works already planned to take place.

It should be emphasised that no reason for concern over the short term integrity of the structure has been identified, however, a medium and long term maintenance strategy is required to ensure the resilience of one of Cambridge's key infrastructure assets. The risks to the public crossing the bridge are assessed to be generally low.

7.1 Recommendations

A staged approach to determining the future resilience of the bridge based on the current condition of the structure should be undertaken to update the assessment carried out in 1996.

Stage 1. Inspection and investigations

- A Bioshield spray was applied in 2016 to kill off the lichen and algae. Now that time has been allowed for this to take effect, the residue should be washed off, any remaining algae and detritus removed and a full Principal Inspection should be undertaken on the cleaned structure. This should identify any areas of corrosion and deformation of the steel work.
- Geotechnical investigations should be carried out and an assessment completed of the underlying materials.
- Concrete and paint investigations should also be carried out at this stage.

Stage 2. Detail Repairs

- Following completion of the inspection and assessment, repair works to be detailed to rectify any defect identified. Any strengthening determined by Stage 1 should also be designed at this stage. The assessment should be reviewed in light of the information gleaned from the inspection.

Stage 3. Repair and Repaint

Appendix A

Technical Structural Review



A.1 Introduction

Victoria Street Bridge (RP 969) also known as Victoria Bridge and Cambridge High Level Bridge is located in Cambridge within the Waipa District, Waikato Region.

Following a traffic analysis report which identified the importance of Victoria Bridge to the long term local plan, Waipa District Council engaged Beca to carry out a study on the structural and geotechnical resilience of Victoria Bridge.

The main report “Victoria Bridge (Cambridge) – Resilience Review Report” is targeted at a wider audience, this appendix is a detailed review of the existing documentation which aims to summarise the existing inspections and assessments with a focus on the structural condition and assessment findings. A separate Appendix B details the geotechnical review.

A.2 Available Documentation

The following drawings and documents have been used for the condition of the superstructure and preparation of this report, note that the majority of pre-1996 information has been taken from the 1996 Bloxam Burnett & Olliver (BBO) report. Various photos were also available of the bridge. Inspections are covered in the next section.

Documents		
Date	Title	Originator
1996	Victoria Bridge – Cambridge – Structural and associated traffic analysis	BBO
1998	Cambridge High Level Bridge Surface areas	Unknown
2009-06-10	Review of Posted Gross Weight Limit (Letter)	BBO
2009-06-10	Confirmation of Bridge Postings (Letter)	BBO
2012-10-17	Victoria Bridge Footpath Widening Detailed Analysis	BBO
2014-05-23	Footpath Widening Defect Record - Lattice Truss (Letter)	BBO
2015-08-14	Bridge Posting Notification 2015 – Victoria St and Baffin St	Beca
2016-08-02	Detailed Structural Inspections (Scope Letter)	Beca

Drawings		
Date	Title	Originator
1905	Road Bridge over the Waikato River, Cambridge NZ (Scan of construction drawings, many dimensions illegible)	Waddell and Hendrick Consultant Engineers
1960	Proposed Strengthening and deck widening - End post and lamp standard details (not constructed)	James North Consulting
1964	High Level Bridge Redecking	North, Swarbrick, Mills and Westwood Consulting
1976	Steelwork - Replacement to Lattice Bracing Drawing	G.A. Hughes & Associates
1976	Steelwork – Plan & Sections	G.A. Hughes & Associates
1976	Elevations – Steelwork Details	G.A. Hughes & Associates
1996	Northern approach height restriction gantry truss	BBO
2014-05-21	Footpath Widening Lattice Truss Defects Tables	BBO
2014-06-16	Footpath Widening As-Built drawings	BBO

A.3 Inspection History

The condition of road bridges are monitored by regular inspections in line with national guidelines set by the New Zealand Transport Agency (NZTA). The bridge inspection policy identifies several grades of inspection and refers to Highways England (HE) “*Inspection manual for highway structures*”, key types are as follows:

General Inspection: Due at a minimum of every two years a General Inspection comprises a visual inspection of all parts of the structure that can be inspected without the need for special access equipment, or extensive traffic management arrangements.

Principal Inspection: Due at a minimum of every six years a Principal Inspection comprises a close examination, within touching distance, of all inspectable parts of a structure. A Principal Inspection should utilise as necessary suitable inspection techniques, access and/or traffic management works.

Special inspection: As agreed or required the purpose of a Special Inspection is to provide detailed information on a particular part, area or defect that is causing concern.

Note that while not explicitly stated within the NZTA policy the HE document to which it refers allows that a longer period between Principal Inspections may be agreed with the Overseeing Organisation providing the proposal is supported by a risk assessment. However Principal Inspection intervals determined through risk assessment shall not exceed twelve years. The Risk Based Inspection Programme Methodology used in the Waipa region would lead to a 6 year Principal Inspection regularity.

The available inspections for Victoria Bridge are shown in the table below. It is reasonable to assume that other inspections have taken place between 1996 and 2010 however these have not been located.

Table 1 Inspections on record

Date	Title	Organisation	Type of Inspection
1996	Structural and associated traffic analysis	BBO	Special
2011-02-10	Bridge Inspection Report	BBO	General
2012-05-25	Victoria & Baffin Condition Report	BBO	General
2013-11-13	General Bridge Inspection Report	Beca	General
2014-06-17	Victoria Bridge Inspection Report	Beca	Special
2016-04-12	General Bridge Inspection Report	Beca	General
2016-05-30	Routine Bridge Inspection Report	NZTA	General
2016-06	Drone inspection summarised in 2016-08 scope letter.	Beca	Special
2016-09	Summarised in the review of lacing bar deflections (2017 Letter)	Avalon (Summary Letter by Beca)	Special
2016-12-04	General Bridge Inspection Report	Beca	General

Note that the type shown above is based on the level of access for the inspector in line with the guidelines rather than the title of the report.

As the main span is largely inaccessible without the use of expensive specialist access equipment it is not known when the last full Principal Inspection (according to the current guidelines) has been carried out on the bridge.

It is likely that the special inspections have been used in place of a Principal Inspection and therefore the details of these special inspections are important in understanding the level of confidence in the current known condition.

The 1996 special inspection used a PA 17000 bridge inspection unit and focussed on critical load carrying members as determined by analysis of the bridge. This is likely the most detailed condition report of the bridge currently available however it is 22 years old.

The 2014 special inspection utilised the scaffolding erected for the footpath replacement to closely inspect the upper areas of the bridge. Details of upper chord lacing defects were also recorded at this time by the BBO team involved in supervising the works.

The 2016 special inspection used roped access equipment to view critical areas identified from a review of drone photography taken earlier that year. The primary focus of this access was to measure the displacement of lacing bars for comparison with the 2014 measurements as well as to spray the structure with an anti-moss and lichen bio shield product to help reduce the ongoing lichen issue. While a general overview of the bridges condition was also conducted no detailed reporting to the level that would constitute a principal inspection of the bridge was carried out.

A.4 Condition

The following condition summary is based on the available inspections detailed above. For further detail and photographs please refer to the inspections.

The main defects that steel bridges commonly present are corrosion and section loss. With any steel bridge corrosion pockets are expected, their extent and severity are determined by inspection and the condition translated into a reduction in calculated capacity through use of a condition factor. These defects are prevented by use of paint systems, drainage systems and removal of vegetation. They are remediated by removal of the corroded areas and either replacing the steelwork or adding new steelwork adjacent to the affected areas.

The condition factor used in the 1996 and 2012 assessments was 0.9.

A.4.1 Primary Superstructure elements

The 2016 inspection is the latest record for the easily accessed areas of the bridge and includes a comment that paint loss and corrosion is present on the beam flanges (back-span beams). There are also photos of corrosion and section loss at some joints including that severe corrosion and section loss present at abutment D on a RHS bottom flange.

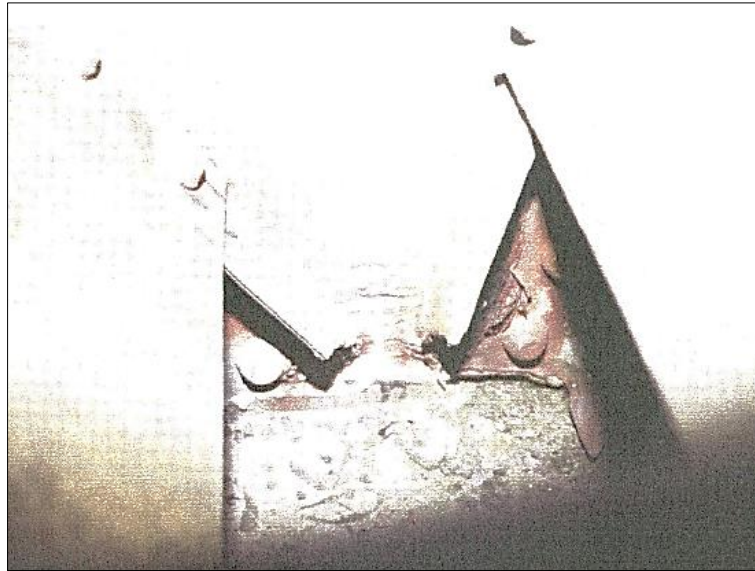
The 2012 inspection noted corrosion in the stanchion channel at pier C (Figure A- 1), this is a heavily loaded member. This defect was also noted in the 1996 inspection which implies not all maintenance actions identified in 1996 were carried through to the subsequent works.

Figure A- 1: Surface corrosion inside lattice column at Pier C (2012).



On the arch members the 1996 report identifies that the nodes are the areas of highest corrosion as they are not exposed to the washing influence of the weather (Figure A- 2). Section loss inside of these node areas is common and extensive parent material replacement has been carried out previously. Material replacement between the lower angle sections and the webs is thought to have been carried out in the 1950's.

Figure A- 2: Corrosion of members inside node area on arch (1996).



Corrosion pockets are observed within the connection points of lacing bars and inner faces of gusset plates. Section loss to channel flanges seems to have been an ongoing issue with replacement having taken place in some locations. Gusset plates tend to be an area of deterioration with several having been replaced in the past.

Cross struts and diagonal arch bracing that have been replaced by RHS components were all noted to be in excellent condition at the 1996 inspection.

The fabricated plate members, transoms and approach span girders were noted as having heavy section corrosion in the 1996 report however this was not thought to cause significant actual section loss. Pigeon guano was thought to be a major contributor to the corrosion on these and other upper elements.

A.4.1.1 June 2016 Unmanned Aerial Vehicle Inspection

Waipa DC conducted an inspection of the Victoria St Bridge main truss spans using an Unmanned Aerial Vehicle (UAV) in 2016. This inspection tested the viability of using UAV's to inspect areas of the bridge that are otherwise difficult to access. The drone obtained video and still imagery of the bridge's steel truss components.

A noticeable amount of algae and lichen build up was observed from the photos (Figure A- 3). The lichen growth tends to be more pronounced at the bottom chord and diagonal members' on the upstream and downstream faces.

Figure A- 3: Drone photography showing algae build-up (2016).



The truss members are generally in good condition. The truss typically exhibited some moderate surface rusting. It is difficult to detect small deflections, bends or kinks in the lacing bars or global deflection defects by visual inspection of the photos.

The following defects were also noted on the truss. The node numbering system for the defects identified from the UAV inspection is based on the bridge construction drawings. This is different to the reference system adopted by BBO for documenting the top chord lacing deflections. Note: "E" denotes eastern truss, "W" western truss.

- One sheared-off gusset plate rivet, node B8-E.
- Inner gusset plate corrosion at interface with vertical member, node B5-E.
- Corrosion in vertical member bottom tie plate, chipped off paint exposing corroded plate, node B4-E.
- Corrosion in tie bracing clevis, node B4-E.
- T6-B6-W and T8-B8-W cross bracing surface rusting in lacing bars.
- Local corrosion at pin-joint plate B7-W.
- Local corrosion at flange of diagonal T6-B7-W- close to B7.
- Inner gusset plate local corrosion B5-W.
- Top end plate of vertical member surface corrosion (connection to transom) T5-W.
- Surface corrosion at bracing connection T4-W.
- Surface corrosion at vertical member end plate B4-W.

A.4.1.2 Lacing bar deflections

The Victoria St Bridge was inspected in May 2014 by BBO as part of the footpath widening works. The defect records from the inspection showed concerns on the observed bent lacing bars typical on the top chord on the central arch truss portion of the bridge. Of particular interest were the deflected lacing bars along the entire length of top chord U3-U4 at the Cambridge end of the eastern truss. Refurbishment work was undertaken on this member (and the mirror member on the western truss) by replacing the deteriorated lacing bars and installing addition batten plates. Measurements taken on the rest of the top chord members found that deflections were not sufficiently significant to require strengthening.

BBO recommended a detailed inspection within two years to repeat the measurements on the top chord lacing bars to identify any increase in deflection. They also recommended considering the need for completing a similar exercise on the remaining lattice members –i.e. deflection measurements to be undertaken on the bottom chord, vertical and diagonal lacing bars.

In 2016 Avalon repeated the lacing bar measurements. The subsequent comparison between the two sets of data by Beca was inconclusive due to lack of correlation between the data sets.

For the lacing bar deflections, BS5400-6 Table 8 specifies a limit to the out-of-straightness deviation at right angles to a plate surface of $L/165$ or 3mm whichever is greater. It was noted that the bar measured in 2016 with high deflections were generally only in discrete locations and majority of bars were observed to be straight or with deflections in the order of 3 to 5mm. Future monitoring was recommended.

Figure A- 4: Drone photo showing lattice elements (2016).



A.4.1.3 Pins

As a three hinged arch the central and springing pins are critical elements for inspection. Due to their locations access is difficult.

The last inspection of the central pins is thought to be in the 1996 inspection, it was noted that the level of corrosion had increased since 1990 and that additional bearing surfaces were recommended due to the ongoing corrosion and lack of maintenance access (Figure A- 5). It is not known if this was carried out as part of the subsequent works.

Figure A- 5: Corrosion to bearing plates at central pin from beneath pin area (1996).



The springing pins have been inspected in 1996 and 2012, corrosion and vegetation growth were present at both inspections (Figure A- 6). When assessed in 1996 the pins were found to pass with a deterioration factor of 0.9 but fail by 20% with a factor of 0.75. The 0.9 factor was considered valid upon inspection but this highlights the need for regular inspection of these pins.

Figure A- 6: Algal growth around pin support at Pier C (2012).



A.4.2 Deck Elements

The 2011 inspection marks the deck as extensively cracked and recommends monitoring. The 2014 inspection identified honeycombing in many areas. Cope reinforcement has been identified as severely corroded (Figure A- 7). The 1996 inspection identified transverse cracking at the same spacing as the asbestos sheeting approx. 1.5m centres. The inspection also noted longitudinal cracking which would correspond with overstressing of the deck. It was not thought that the cracking had deteriorated between the 1990 and 1996 inspections.

Figure A- 7: Exposed reinforcement and honeycombing in deck (2014).



Due to corrosion of the stringers top flanges there has been some separation between the steel superstructure and the concrete deck, this includes concrete broken around the shear connections effectively reducing the transfer of shear between the elements (Figure A- 8). This was also noted in the 1996 inspection and poor deck waterproofing was suggested as a cause.

Figure A- 8: Concrete breakdown around shear connectors (2014)



Detritus was noted in expansion joints and reflective cracking in deck surfacing. These are routine maintenance issues common with most bridges.

A.4.3 Substructure

A detailed review of the geotechnical status of the bridge foundations can be found in Section 6 of this report. Minor historical cracking in the wing walls has been noted in most of the reports but does not appear to be deteriorating. The 2012 report noted that uneven concrete was present beneath the western springing pin at pier D.

A.4.4 Paint System

It is not known when the bridge was last painted in its entirety however the 2011 inspection implies that this was done in 1999 following the recommendation in the 1996 report. The bridge was also painted in 1978 and washed in 2010.

The 2016 inspection is the latest record for the easily accessed areas of the bridge and includes a comment that paint loss present on the beam flanges (back-span beams).

The 2014 survey of the upper beams noted a breakdown of the paintwork and spot corrosion blistering on the webs and beams (Figure A- 9). It was highlighted that this was present on many of the beams at this level and that general corrosion was also present.

Figure A- 9: Typical corrosion blistering on transverse beam (2014).



There is no recent close up survey of the remainder of the bridge paint system. Widespread algal growth has the dual issue of preventing the paintwork from being inspected remotely (through the UAV footage) as well as through degrading the paintwork from root growth. The 2016 bio shield application should have reduced this issue however it will not have been eliminated. The 1996 report noted that the lower areas were generally in good condition other than the algae due to their exposed nature causing a natural washing of the structure. Due to the age of this report a principal inspection should be carried out to determine a reliable condition factor.

A.4.5 Asbestos

While no documentation has been found that refers to testing, there is documentation that refers to the permanent formwork beneath the deck as asbestos boards. If in good condition and left in place the presence of asbestos is not an immediate issue for the general public. However, an asbestos management plan should be in place for the structure and should investigation or any future maintenance work be carried out that may disturb the asbestos care will need to be taken that a suitably qualified contractor is engaged to control its removal.

A.4.6 Footpath and Railings

Replaced in 2014 the subsequent 2016 inspection noted that these elements were in good condition. Note that the damage caused to the railings in 2015 by an errant truck on the bridge was repaired.

A.4.7 Services

Victoria Bridge is a major service crossing for Cambridge, a 250mm diameter water main is supported beneath the deck from one of the main longitudinal girders passing through the transoms.

All other services are supported beneath one of the two footpaths lying on a horizontal support at the footpath support frames and on intermediate support saddles hung from the FRP support angles.

In 2014 these services consisted of;

Beneath western footpath:

- 4 No. Chorus 100mm diameter ducts containing copper wires.
- 1 No. Chorus 32mm diameter fibre optic cables.
- 1 No. Ultrafast Fibre Ltd 100mm diameter ducts containing fibre optic cable.

Beneath eastern footpath:

- 1 No. Waipa district Council 32mm diameter lighting duct.
- 1 No. Vector Gas 100mm diameter duct containing 75mm diameter gas main
- 1 No. Vector Gas 150mm diameter duct containing 100mm diameter gas main.
- 1 No. 100mm diameter empty duct.

The condition of services have been recorded as satisfactory and those beneath the footpaths will have been handled during the 2014 repairs where any defects are likely to have been fixed if found.

A.4.8 Existing Maintenance Actions

A combination of the maintenance actions from the 2012, 2014 and 2016 inspections are listed below. It is considered that actions before this period may no longer be valid. It should be noted that these are recommendations by the bridge inspector and must be compared against similar recommendations for other bridges in the region and budget assigned accordingly. Hence it is not uncommon to see duplication between several reports.

2012 Inspection

- Installation of a netting system to prevent pigeons from roosting on the structure.
- Vegetation clearance on the north bank to help prevent damp conditions perpetuating around the northern portion of the substructure, around pier C, often in the shade.
- It was recommended that the Council should plan for expenditure in the region of \$1m in their 10 year plan, programmed for around 2017, to cover the cost of installing a new protective paint system to the steelwork.
- Resurfacing within the following 2 years was recommended. The application of an epoxy membrane sealing coat to the concrete deck is recommended prior to application of any surfacing, to preserve the deck reinforcement.
- It was recommended that Waipa DC consider whether the Victoria Bridge would benefit from a principal inspection to ensure that all defects are identified and maintained preventatively.

2014 Inspection

- Programme a detailed inspection of the bridge within the following two years, in order to verify the condition of all lattice members in the lower levels of the arch trusses and cross bracing, (as recommended by BBO) and establish future maintenance painting requirements for these elements.
- Re-painting of the underdeck steelwork is required in the medium term (2 – 6 years), in order to arrest further deterioration of the steelwork, due to corrosion.
- Treat the exposed reinforcement on the deck copes and apply a concrete repair, along the length of the deck, to enhance the cover to the deck reinforcement. This could be carried out at the same time as the painting, although it is a lower priority.
- Consideration should be given to the installation of inspection walkways below the deck, in order to simplify future inspection and maintenance of the structure.
- Consideration should also be given to installing netting beneath the deck to provide a more effective bird exclusion zone.

2016 Inspection

- Crack seal surfacing at deck panel joints. Remove detritus from main expansion joints.
- Remove corrosion and repaint severe corrosion and section loss at Abutment A LHS bottom flange.
- Principal Inspection and Corrosion Management Plan required to determine optimal painting strategy.

A.5 Previous Assessments and Restrictions

Traffic restrictions on bridges are known as postings, the posting of bridges is reviewed on an annual basis by the incumbent engineering consultancy. These postings are updated if significant deterioration or strengthening works have occurred since the last assessment of the structure.

An axle weight restriction has been in place since at least 1950 where a 5 ton limit was in place alongside other restrictions on logging trucks and stock loading. The 1950 Ministry of works document (referred to in 1996 report) noted that the bridge was satisfactory for H20-S16-44 lane load on the deck centreline or H20-S16 truck anywhere on the deck.

When the timber deck was replaced with concrete in 1964 the concrete deck increased the dead load (weight of structure) on the bridge correspondingly reducing the allowable traffic load. The designer of the concrete deck has stated that it was designed to only take cars hence the very low thickness of 95mm (normally ≥ 150 mm at this time). The bridge loading restriction was reviewed and the following posting applied:

2000kg gross weight, 25km/hr speed restriction, 3m height, 2.1m width.

Due to high numbers of heavy vehicles ignoring the imposed restrictions gantries were installed to prevent large vehicles using the bridge. A full assessment of Victoria Bridge was carried out in 1996 by BBO and concluded that this posting be retained. The critical component of the bridge was determined to be the bridge deck.

The posting has since been reviewed by BBO in 2009. The reason for this review was that modern SUVs were becoming more popular as family vehicles and SUVs tend to weigh in excess of 2000kg. While the council wished to enforce the postings on the bridge to ensure the longevity of the structure they did not want to limit the average family vehicle from crossing.

BBO did not carry out a full assessment of the structure but instead compared SUV loading to the loading limits from the existing 1996 assessment, concluding that the steelwork was capable of supporting the 3 tonne loading. As the deck governed the previous assessed posting it was therefore considered sufficient to only carry out detailed assessment on the concrete deck to determine if the posting could be raised. The outcome of this review determines the current posting:

3000kg gross weight, 30km/hr speed restriction, 3m height, 2.1m width.

Note that this posting was recommended contingent on an annual inspection of the deck, preferably with the removal of permanent formwork in an area easily accessible.

As part of the 2014 footpath widening works the steelwork was assessed by BBO in 2012 to confirm that the footpath could be supported. This was carried out using a 2D structural analysis model for a single side of the bridge and did not include analysis of the deck or seismic and wind loading because the widening of the footpath did not increase the area of the bridge subject to wind or the dead load subject to seismic events. The calculations referred to the previous 1996 analysis for these effects.

On 21 October 2015 it was reported that a 42 tonne truck passed over the bridge despite the restrictions imposed. An emergency structural survey was undertaken and the bridge was temporarily closed. It is understood that WPC and Beca were both present and the results of the visual survey were conveyed verbally on site. The structure was reopened following the survey and railings damaged in the incident were subsequently replaced.

A.5.1 1996 Assessment

The last full assessment of Victoria Bridge was carried out in 1996 by BBO. Combined with the 2009 deck review this assessment forms the basis of the current posting.

The 1996 assessment was carried out in two stages, firstly before the condition inspection and metallurgical testing was carried out (to identify areas to concentrate inspections) and secondly after the data had been retrieved. The methodology of the assessment did not change but the inputs varied in the second analysis to reflect the inspection and testings' findings. The findings from the final analysis are summarised below.

A.5.1.1 Methodology

The whole length of the bridge was modelled in 3D using a program called "Microstran Version 5.5". The footpaths, deck and deck supporting stringers were omitted from the model. The bracing and transoms beneath the deck were modelled along with the arch truss members and primary beams on the back-spans.

Full assessment traffic loading (0.85HN-72) was applied to model Class 1 vehicles and the maximum demands compared with calculated capacities. The output was a tabulated detailing the demand over capacity percentage for each member under this loading.

The percentage of Class 1 live load representing nose to tail cars was also calculated and compared to those calculated for the members this value was approximately 40%.

The deck was analysed using hand calculations and Pucher Charts.

A.5.1.2 Loading

The modelling software was used to apply self-weight to each of the elements based on their cross-sectional area. The dead loads of the deck and footpaths were applied as concentrated loads to the transoms at the beam seating locations.

A single lane of full HN-72 live loading was applied to the structure both centrally and at maximum eccentricity. Several load configurations were used to determine the maximum live load effects. The live loads were also applied to the transoms as concentrated loads.

A 4kPa footpath live load was used (documented in the 2012 report).

A.5.1.3 Inputs

Following metallurgical testing the steel was identified as medium carbon steel with a yield strength of 200MPa.

A load factors of 1.2 was applied to the dead load. Traffic loading of 0.85HN-72 with an impact factor of 1.3 and a load factor of 1.9 was applied.

The capacities of the members were determined using BS 5400: Part 3 which applies partial factors equivalent to a 15% reduction in capacity.

The condition factor used to account for the deterioration of the structure was 0.9.

A.5.1.4 Posting Conclusions

The critical steel members were found to have a capacity of 43% Class 1. This was less than the 40% Class 1 calculated for the nose to tail cars.

The deck capacity was found to be 18% of the capacity of a standard axle, and the deck elements governed the bridges overall posting. Flexure rather than shear was identified as the limiting behaviour and a 1500kg axle was determined as the safe limit.

A.5.2 2012 Assessment

The 2D analysis carried out in 2012 for the footpath widening confirmed the ability of the steel structure to withstand traffic loads and the new footpath loads under the current traffic restrictions. The scope of this assessment did not include analysis of the deck or seismic or wind loading. As the 1996 assessment identified traffic loading as the critical load case for the steelwork the 2012 analysis was used as the basis of the steelworks resistance to traffic loading.

The analysis used two software packages, Microstran and CSI Bridge. The main arch span was modelled in 2D with hand calculations completed for the back-spans and the footway itself.

Dead loads from the deck and footway as well as traffic loads were calculated for each transom and applied at the relevant nodes as concentrated loads. Dead load was factored by 1.25 to account for bracings and connections.

The traffic vehicle load used was based on two 1.5T axles at 5m spacing. The footway loading was calculated based on the span and applied accordingly.

Lateral stability of the axial truss member was checked by applying 2.5% of the member's vertical load in line with the 1956 Bridge Manual and was found to be compliant.

Under the current loading regime the assessment found the structure to comply with standards with the exception of the 65ft approach span girders. These girders were found to be 119% utilised. The explanation for this difference to the 1996 assessment was that the assumed lateral restraint for the girders differed. To remedy this non-compliance with standards the assessment suggested installing shear connectors through the top flange of the girders and into the concrete deck.

A.5.3 Fatigue

Fatigue was not considered in the 2012 Assessment therefore the latest fatigue analysis was in 1996 and is summarised below. The analysis was based on BS540 Part 10 and concluded that the fatigue issues were unlikely to occur due to the previous loading history and continuation of the ban on heavy vehicles. It was considered unlikely that fatigue considerations would govern provided that certain minor remedial work was undertaken. This remedial work focused on the transoms where replacement angle sections had incomplete butt welds. It was noted that these should be upgraded. Inappropriate modifications were also found on the stringers, however, these were noted as not to effect the bridge to a significant extent.

A.5.4 Wind and Seismic Loading

Neither wind nor seismic loading was considered in the 2012 Assessment therefore the latest analysis of these was in 1996 and is summarised below.

A.5.4.1 Wind

The wind load case was found to be more critical than seismic loading due to the high area of steel compared to the relatively low self-weight. This load combination governed for some of the bridge members to the point where some members were stressed by the wind load case more than double the amount under traffic loading. It was however found that all members complied with assessment standards.

The assessment report does not detail the calculations undertaken for wind loading. It is therefore not possible to determine the validity of the methodology used. The standards to which the Bridge Manual refers have changed since 1996, while a detailed comparison has not been made there does appear to be sufficient changes in the standards to warrant a review of this load case, particularly due to the local topography (a gully funnelling potential wind effects).

A.5.4.2 Seismic

Seismic loading was based on the 1994 Bridge Manual. A C_d of 0.25 was calculated using a period of 0.9s and ductility factor, μ , of 1.25. A 0.25g transverse acceleration was then applied to the structure. This was found to be less than the wind load effects for the majority of members.

Seismic linkages have been added to the structure since the 1996 report identified the lack of linkages for the back-spans.

The current equivalent static force method detailed in the Bridge Manual is different to that used in the 1996 calculation. However, while there are some differences the horizontal design seismic coefficient to current standards is similar to that used in 1996. For this reason the analysis is considered to still be valid.

The 2012 calculations note that, should the structure be considered to provide a lifeline route following a major seismic event, the Council may wish to carry out a more detailed analysis.

A.6 Discussion of Structural Considerations

Victoria Bridge is a key route both for vehicle and pedestrian traffic as well as critical services connecting the two parts of Cambridge divided by the Waikato River. In order to determine a long term strategy for the town's infrastructure it is important to know how vulnerable this key route is and what expectations can be made of the bridge's service life and upcoming maintenance requirements.

A.6.1 Structural Vulnerabilities

The last detailed structural analysis and assessment of the whole structure was carried out in 1996 where the remaining service life of the structure was estimated to be in excess of 30 years. From the available inspections the bridge is considered to be in good to fair condition with defects in line with the age of the bridge.

As is common among steel structures the key issues of structural integrity for the superstructure are associated with corrosion and section loss of elements. It is known that Victoria Bridge does have corrosion present, some severe, particularly around nodes and the central pins.

The structure does not have a full Principal Inspection on record and the majority of the bridge has not been inspected in detail for many years with a reliance on the limited special inspections to flag any issues with the condition or defects. The lack of a substantial condition survey and associated maintenance strategy raises the risk both to the structure's integrity and to unforeseen maintenance costs. As the assessment relies on accurate knowledge of the condition, the outcomes of an inspection may have an impact on the assessed capacity of the bridge requiring repair and strengthening work to be undertaken.

While the current seismic analysis is considered to be valid the 2012 calculations note that, should the structure be considered to provide a lifeline route following a major seismic event, the council may wish to carry out a more detailed analysis. However, as there is an alternative route via State Highway 1 the Victoria Street crossing is not considered a lifeline route.

A.6.1.1 Deck capacity risk

The 95mm depth of the concrete deck slab governs the capacity of the bridge and means that the integrity of the deck is particularly susceptible to any defects that may occur.

The honeycombing, cracking and reinforcement corrosion observed in the concrete deck are highlighted as a potential risk and need to be investigated. The concrete investigations should include the removal of asbestos sheets over a land span to allow for future observation as well as crack mapping and concrete coring to the deck to determine strength and to examine for evidence of defects.

Appendix B

Technical Geotechnical Review



B.1 Introduction

Victoria Street Bridge (RP 969) also known as Victoria Bridge and Cambridge High Level Bridge is located in Cambridge within the Waipa District, Waikato Region.

Following a traffic analysis report which identified the importance of Victoria Bridge to the long term local plan, Waipa District Council engaged Beca to carry out a study on the structural and geotechnical resilience of Victoria Bridge.

The main report “Victoria Bridge (Cambridge) – Resilience Review” is targeted at a wider audience, this appendix is a detailed review of the geotechnical resilience based on existing data and a recent site inspection. A separate Appendix A details the structural review.

B.2 Available Geotechnical Information

A review of geotechnical information held by Beca in the area include the following past studies:

- Beca Ltd (2017) Cambridge C1 C2 and C3 Plan Change Liquefaction Hazard Assessment.
- Beca Ltd (2015) Cambridge Pipe Bridge.
- Beca Ltd (2012) 12 Alpha Street Cambridge Pre-Conceptual Geotechnical Assessment.

These investigations are within areas underlain by similar ground conditions but are all too far away to be specifically applicable to assess ground conditions for Victoria Bridge.

A river boat inspection of the river banks was completed on 20 February 2018. A site walkover was completed on 10 July 2018.

B.3 Site Description

Victoria Bridge in Cambridge spans 142m across Waikato River adjoining Cambridge and the suburb of Leamington. The 88.4m central span is supported by two steel arches with no supporting piers in the river.

The northern river bank varies between 49mRL and 52mRL and is around 32m above river level, sloped at around 1.2:1 (Hor:Ver). The southern river bank is at approximately 55mRL and is around 37m above river level sloped at around 0.8:1 (Hor:Ver). Refer to Figure B- 1 and Figure B- 2 for recent photos of the river banks.

The northern approach and northern abutment to the bridge is supported on an approximately 6m high embankment above surrounding ground level, set back around 10m from the crest of the river bank slope.

The northern and southern arch foundations for the bridge are located on steeply inclined river bank slopes. Each arch is supported by separate concrete foundation. Scanned copies of the original drawings (Waddell & Hedrick, 1905) indicate that each arch foundation is around 4.3m deep, 3.3m wide and 6.0m long.

The bridge abutments and spans on either side of the main arch appear to be supported by shallow concrete foundations. The abutments also retain the approach embankments between 4.5m and 6m high.

The river bank below the southern abutment foundation has a concrete treatment, as shown in Figure B- 1. This is consistent with a documented repair completed in 2012 following instability of the lower river bank slope (BBO, 2012). The repair was reported to comprise soil nails and shotcrete plus erosion protection works. No design drawings for the works have been reviewed.

The Waikato River varies in level controlled by discharges from the Karapiro Dam. Typical river levels are expected in the range of 16mRL and 20mRL.

Areas beneath and surrounding the bridge are currently well vegetated.

Figure B- 1: Northern bridge abutment slopes.



Figure B- 2: Southern bridge abutment slopes.



B.4 Geology

Published geological maps for the area (Figure B- 3) show the surficial sediments to be laminated, commonly cross-bedded, fluvial sands and gravels, fragments of pumice and ash and some primary ignimbrite material of the Hinuera Formation, Q1a (Edbrooke, 2005). Localised deposits of pumice sand, silt and gravel alluvium belonging to the Taupo Formation have also been noted in the wider area.

Walton Subgroup sediments comprising late Pliocene to mid. Pleistocene (2.4 million years to 60,000 years) volcanic ash, ignimbrites and redeposited volcanic silts, sands and gravel are expected to underlie the Hinuera and Taupo Formations.

Boat inspections of the lower river bank slopes suggest than Walton Subgroup materials are present in the lower slopes.

Figure B- 3: Geology of the Victoria Bridge.



B.5 Geotechnical Constraints

This section presents potential geotechnical constraints based on a review of available information and a site visit. In most instances, further assessment is needed to better understand these constraints and their potential effect on the bridge.

B.5.1 Liquefaction

The natural materials within the river bank supporting the bridge have the potential to liquefy in a moderate to large earthquake event if saturated. Liquefaction effects include a significant strength loss that could make any slope instability in an earthquake more significant.

Ground water levels encountered in studies carried out by Beca in the Cambridge area suggest the site surrounding Victoria Bridge may have locally perched ground water and possibly high ground water levels. Proximity to the river bank is expected to locally lower ground water levels.

Liquefaction could occur if loose sandy layers coincide with perched or elevated groundwater levels. Further assessment is needed to assess the liquefaction potential for the site.

B.5.2 Slope Instability

The existing river bank slopes are unlikely to meet typical design standards for slope stability under static or earthquake loading. Potential slope instability may include shallow instability of the steep river banks as a result of elevated ground water levels or surface water run-off, or larger deep seated slips under seismic loads.

River bed degradation will also tend to steepen the lower river bank slopes over time and may increase the risk or instability of the lower river bank slopes. This is discussed further below.

Slope instability losses may undermine the bridge foundations over time, reducing their load carrying capacity. The stabilisation treatment below the southern arch foundations was likely constructed as a result of past stability concerns for the river bank supporting the foundation.

Some evidence of localised shallow instability was noted while on site, including documented remediation of the southern abutment slope in 2012. The existing vegetation cover indicates that larger scale slope instability has not occurred in recent times.

B.5.3 River Erosion

The Waikato River is in a state of degradation (Smart, 2005). This study estimated that future river bed erosion may be in the order of 1.5m in the next 50 years or around 30mm/year on average. This erosion rate may be locally lower here where Walton Subgroup material are encountered at river level.

Steepening of the river banks will likely continue as a result of any future river bed degradation. Future small scale instabilities associated with the loss of material at the toe of the slope are likely to occur periodically. The arch foundations on both sides of the river may be vulnerable to loss of support as a result of slope instability of the lower river bank triggered by ongoing river erosion.

B.5.4 Internal and Surface Erosion

Localised signs of surface erosion were noted at the northern bridge abutment, particularly associated with stormwater drainage discharge locations. This is not anticipated to directly affect the bridge foundations, though could reduce support or trigger slope instability over time.

No evidence of internal erosion was observed, though such features are not typically evident until failure occurs and are obscured by vegetation cover.

B.5.5 Faulting

There are no mapped active faults through or near the site as mapped in GNS active fault database (Geological & Nuclear Sciences, 2015). The nearest active fault is the Kerepehi Fault, a normal fault running between Matamata north into the Firth of Thames.

The understanding of faulting along the Waikato River has developed in recent times with new faults identified, often associated with bends in the river or gully features. These faults are often not classed as active, though can result in areas of locally weaker ground conditions.

B.6 Recommendations

Based on the review of available data and the site inspection completed, future geotechnical assessment is recommended to include:

- Complete a geotechnical investigation to gather information on the soil and groundwater conditions at the site.
- Develop a representative ground model.
- Complete a liquefaction assessment of the river bank materials under ULS and SLS load cases.
- Complete river bank stability assessments under sustained and extreme load cases.
- Review the potential for internal erosion to occur within materials supporting the bridge.
- Assessment of the geotechnical capacity of existing bridge foundations.
- Review of soil structure interaction effects and assessment of likely foundation performance under normal and extreme conditions.
- Outline remediation concepts, if appropriate, to reduce geotechnical risks.
- Document the findings in a Geotechnical Assessment Report.

Geotechnical investigations for the bridge are recommended to include the following:

- Machine boreholes at both abutments to between 30m and 40m depth (sufficient to confirm materials in the lower river banks).
- Two seismic cone penetration tests (sCPTs) to approximately 20m depth (or refusal) to assist in liquefaction assessment.
- Installation of two standpipe piezometers to allow measurements of ground water. These will require some ongoing monitoring.
- Laboratory testing to refine the liquefaction risk.
- Buried service checks and traffic management at all test locations.
- Roped or boat access inspection of the main arch foundations and check of the condition of stabilised southern slopes. This may require some vegetation clearance.

Requested additional inputs include:

- Design drawings for the remediation works completed in 2012 on the slope below the southern arch foundations.

B.7 References

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