Victoria St Bridge(RP969) - Main Arch Bridge Structural Assessment Report

Prepared for Waipa District Council
Prepared by Beca Limited

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Revision History

Revision Nº	Prepared By	Description	Date
A	Liam Edwards	Draft for review	19/03/2021
B	Liam Edwards	Updated assessment based on June 2021 site measurements	24/09/2021
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Document Acceptance

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on behalf of	Beca Limited		

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Executive Summary

Victoria Street (RP 969) Bridge is a four span steel structure over the Waikato River. The bridge was designed and constructed in 1905 and opened to public in 1907. The bridge is 141m long, comprising an 88m long main arch span with steel girder approach spans on either side.

Waipa District Council (Council) have commissioned Beca to undertake a structural assessment of the main arch span and central pin connection as a result of the recommendations from the 10 August 2020 Condition Report prepared by Beca.

The structural assessment of the main arch revealed that the section capacities and demands on the structure are lower than the previous assessments (circa 1996 and 2012) undertaken and this has been determined from site measurement of critical elements in June 2021. Further, the previous assessment may not have commented on the gusset connection capacity or main member lacing effectiveness, which are reported on in this report.

Under the current operating case, no strengthening of the bridge is considered to be required, and specific findings relating to the assessment are outlined below.

It was found that:

- The lacing of the main arch members do not meet the slenderness requirement. The lacing strength requirement of AASHTO* is not satisfied, but is for NZS3404* and AS/NZS 5100.6* and for the assessment loading this is considered an acceptable standard.
- The arch splice gusset connection was found to pass the assessment standard.
- The critical member for the main arch was member L2-L3, with the remaining bottom chord members only slightly less critical.
- The main arch has sufficient capacity to accommodate the ULS load combination for case 1 loading, which is a continuous stream of two lanes of light vehicles simulated as two lanes of 1.5 tonne axles at 5m centres.
- The main arch does not pass the assessment standard to accommodate the ULS load combination for 0.85HN loading (case 2 in this report). It is noted that these loads are not allowed on the bridge.
- The strengthening plates for the pin are adequate to accommodate the total load for the ULS case 1 (Cars), but not ULS case 2 (0.85HN). They were found to provide an alternative load path to the original bearing plates.

For future management of the bridge, steelwork repairs should be developed and undertaken on areas of significant corrosion, and the bridge should be repainted to maintain its corrosion protection system and prolong the life of the bridge.

**Refer to section 3.1.1 for full standard references*

1 Scope of Assessment

The overall scope of the assessment is broken into three portions and outlined as below.

1.1 Live load Evaluation of Main Arch

The main arch assessment scope is to carry out structural assessment for the entire main arch of the bridge in accordance with the Waka Kotahi NZTA Bridge Manual for dead and live loads.

The assessment process will review main member elements only and not connections (with the exception of the one connection noted below).

1.2 Central Pin Assessment

The scope is to assess and compare the capacity of the central pin connection, for both the original arrangement (1905 Original Drawings) and the strengthened arrangement (1998 Strengthening Drawings) and to identify if the strengthened arrangement can carry all loads in insolation of the original details.

1.3 Gusset Connection Assessment

The gusset connection assessment scope is to assess the capacity of one gusset connection to identify if any further assessments should be undertaken on other connections. The gusset connection chosen is based on the demand outputs from the model, and engineering judgement was used to determine the likely critical gusset connection for assessment.

2 Bridge Background Information

Victoria Street (RP 969) Bridge is a four span steel structure over the Waikato River. The bridge was designed and constructed in 1905 and opened to public in 1907. The bridge is 141m long, comprising an 88m long main arch span with steel girder approach spans on either side. The Leamington approach span is 19.8m while the Cambridge approach consists of two spans (19.8m and 12.8m). The main span is founded on the steep river banks. The two end spans are supported on mass concrete abutments.

The original deck of the bridge was timber and there was no footpath on the bridge. The old timber deck has been replaced with a 95mm thick concrete deck, and a 1.5m FRP footpath has been added to each side of the bridge since the bridge was originally constructed.

The steel elements under the bridge were last painted in 1999.

The bridge currently has a weight restriction of 3 tonnes gross, a height restriction of 3m and a width restriction of 2.1m for each lane.

For further details refer to the Stage 1 Resilience Review Report, dated August 2018.

2.1 Previous documentation and history of bridge

2.1.1 Previous Assessment of bridge

In 1996 a structural assessment was undertaken on the bridge which had the following key findings:

- The analysis indicated that the limiting structural steel component can safely carry about 43% of Class I traffic loading. The limiting element was element L2-U2.
- The present concrete deck is very thin in comparison to normal concrete decks and as such has a very low load carrying capacity. The analysis indicated a limitation of 1500kg per axle is appropriate.

- The bridge cannot be viably upgraded to take two unrestricted lanes of Class I traffic.

2.2 As-built information – original details

The record drawings available on file for this bridge structure are very difficult to read. A brief search was carried out at Waipa District Council, however, we were unable to locate any original hard copy drawings or Microfiche for the bridge. Requests were made to the previous bridge assessors (Bloxum Burnett & Oliver Ltd (BBO)), and also to WSP; however, we were unable to locate any drawings which were more legible than the record drawings available.

A brief search on NZ National Archives (NZNA) website located a more legible elevation drawing of the original bridge superstructure; however, some of the member sizes on this elevation appear to be different to the record drawings available. Neither of the drawings are clearly labelled with a date, so it is not clear which drawing is most current. The majority of the section sizes on the two drawings are the same except for three of the members which appear to consist of larger/thicker individual elements compared to the NZNA drawing. We have assumed the record drawings take precedence over the NZNA drawing.

Section properties for the various built up sections were adopted from historic steel section references as appropriate. One reference used dates back to 1907 from the Bethlehem Steel Company.

To determine the member section sizes we used the drawing found from National Archives as a base, and compared this to the record drawings available. In summary the only members that appear to be different to the drawing from National Archives are the bottom two chord members near both base supports and the segment of bottom chord on either side of the central pin. The assumed section sizes from these drawings were used to determine the member capacities reported in revision A (19/03/2021) of this report.

Subsequently, in June 2021, Lumen carried out a rope access visual inspection of the main truss bottom chord components. A summary of their inspection, including measured plate sizes, is provided in Appendix A. The arch member capacities, provided in section 4, have been updated to reflect the measured plate sizes.

We note that our interpretation of the member section sizes produced lower axial compression capacities to the historic BBO assessment of the structure using the site measurements. Using the section sizes obtainined from the national archives, our results were similar to the historic BBO assessment.

3 Assessment Methodology and Loading

3.1 Methodology of assessment

3.1.1 Standards used in the assessment

The live load evaluation has been undertaken in accordance with the latest edition of the Waka Kotahi NZTA Bridge Manual, Version 3.3, and further referred to in this report as ‘Bridge Manual’.

Standards used are noted below:

- NZS3404:Part 1: 1997 – Steel Structures Standard, has been used and is further referred to as NZS3404.
- AS/NZS 5100.6:2017 - Bridge Design Part 6 Steel and Composite Construction, has been used is further referred to as AS/NZS 5100.6
- American Association of State Highway and Transportation Officials Manual for Bridge Evaluation (AASHTO MBE), 2nd Edition, 2011, including 2016 Interim Revision has been used, and is referred to as AASHTO MBE.

- American Association of State Highway and Transportation Officials Load and Resistance Bridge Design Specifications (AASHTO LRFD), 8th edition, 2017 has been used for the capacity assessment for all main arch members and further referred to as AAHSTO within this report.

3.1.2 Assumptions of assessment

This assessment only covers main members of the arch and does not cover connections (except the pin and gusset connection described in sections 5 and 6), bracing elements, the local deck capacity, footpath, transoms and minor elements.

3.2 Loading

3.2.1 Dead load (DL)

Self weight for all steel elements is taken from the member properties assigned. 20% additional weight on top of primary member sizing has been used to allow for gusset plates, rivets, and other minor connection steelwork.

Steel Stringers

Total allowance for longitudinal steel stringers is taken as 6.4kN/m.

Concrete Deck Slab

Total area of concrete deck slab is adopted as 0.643m^2 per metre with a unit weight of 24kN/m^3. Resulting in a UDL 15.43kN/m

Utilities allowance on concrete deck

Utilities allowance is taken at 0.5KPa over the centre 5m wide carriage.

Footpath Deadload

Footpath deadload is taken 3kN/m per side of bridge. which allows 2kN/m for the 2014 weight of footpath and nominal utilities ducts supported by this, and 1kN/m per side for the handrails.

Surfacing

Surfacing is taken as 20mm thick over the central carriage way assuming a unit weight of 22kN/m2. This was taken from the previous BBO assessment report.

3.2.2 Traffic Live load (LL)

The bridge was originally designed for three types of live load; livestock load, a traction engine pulling two wagons and an electric tram car. Refer to the 1996 assessment report which outlines these loads. These loads are not relevant to the current assessment and were not considered.

Previous assessments considered the below traffic cases also.

This assessment considers two traffic loading cases:

Case 1 – Two lanes of car loading

A continuous stream of two lanes of light vehicles with 1.5T axles has been simulated as a uniform load of 5.9kN/m/bridge, or 2.95kN/m/lane. This was also used by BBO in the Victoria Bridge Footpath Widening – Detailed Analysis report dated 2012 as part of the footpath widening assessment works.

This case is the current bridge operating case.

Case 2 – Single lane of 0.85HN

For the purposes of assessing the bridge to current standards, 0.85HN (Class 1) is applied to one lane on the bridge. Two lanes of HN load element in the bridge manual cannot physically fit on the narrow bridge deck of approximately 4.8m wide and therefore only one lane is considered.

0.85HN loading has been assumed for the assessment which covers the majority of legal vehicles and is used in the evaluation of existing structures as required by the latest edition of the Bridge Manual.

The traffic load case is run over the length of the bridge to determine worst case demands on the main members of the arch.

This case was used to check our assessment outcome with that of previous assessments undertaken.

Impact Factor for live load cases (I)

An impact factor of 1.1 is used in the assessment for all main arch elements due to the 88.4m span of the bridge. The live loading considered is for a ‘traffic jam’ scenario and therefore the traffic will be moving slowly, which also justifies a low impact factor selection.

3.2.3 Footpath Live load (FP)

Footpath loading of 2.06KPa for each footpath has been considered on each side of the bridge. This results in a total UDL of 3.1kN/m for each footpath. This has been considered across the entire structure at once.

3.2.4 Other loads not considered

Wind loading, seismic and thermal and fatigue loading has not been considered in this assessment as these are not part of the current scope.

Foundation loads, and earth pressures are not considered in this assessment, as the scope of this assessment is limited to the steel superstructure of the main arch only.

3.2.5 Load combinations and Load Factors for assessment

Load combinations are in accordance with Table 3.2 of the Bridge Manual

Load factors are taken from Section 7 of the Bridge Manual which states load factors to be used for bridge assessments. These are stated in the below table.

Table 3-1 – Load factors

Load	Load Factor
DL -All Dead Load (Including SDL)	1.2
LL*I - Vehicle Live load (Cars or 0.85HN)	1.9*I
FP - Footpath Live load	1.76

3.2.6 Steelwork assessment approach

The previous 1996 assessment used BS5400:Part 3 ‘Code of Practice for Design of Steel Bridges’ to assess the structural steelwork. However for the purposes of this assessment AASHTO has been used to determine the capacity of the elements.

3.2.7 Material properties

Steelwork material properties for the assessment are based on a yield strength of 200MPa, based on the 1996 assumptions and yield strength testing undertaken. The report which contains this information is the 'Victoria Bridge – Cambridge, Structural and Associated Traffic Analysis – September 1996'.

4 Main Arch Assessment

4.1.1 Members being assessed

The main arch members investigated include all members indicated in Figure 4-1 namely:

- Top chord elements U0-U6
- Bottom chord elements L0-L6
- Vertical elements U0-L0 to U6-L6
- Diagonal elements U0-L1 to U5-L6

4.1.2 Strength Reduction Factors.

The strength reduction factor adopted for the axial capacity of the elements is 0.9 as per NZS3404 and AS/NZS 5100.6. This is slightly conservative compared with AASHTO which allows a strength reduction factor = 0.95 for steel compression members.

4.1.3 Condition Reduction Factor.

A 'condition factor' of 0.9 was applied to all calculated capacities on the basis outlined in the following points:

- We have not done a thorough inspection of all of the members and member connections. We note the detailed inspection undertaken by Lumen in 2019, and their recent inspection in June 2021, indicated some areas of minor and severe corrosion.
- As the bridge was built in 1907 and has varying levels of corrosion and general degradation, and the bridge is considered to be somewhere between good/fair (condition factor 1.0) and deteriorated (condition factor 0.8).

4.1.4 Member sizes used in assessment

Bottom chord members

All bottom chord members appear to have 2.25" x 3/8" single lacing bars on the bottom. The following table summarises the bottom chord members, based on the site measurements provided by Lumen, that were used for this assessment.

Table 4-1 – Bottom chord member sizes used in assessment

Member	Top flange	Webs	Top angles	Bottom angles
L0-L1	14" x 5/16"	2 No. 17" x 5/16"	2 No. 3.5" x 3.5" x 3/8"	2 No. 6" x 6" x 5/8"
L1-L2	14" x 5/16"	2 No. 17" x 5/16"	2 No. 3.5" x 3.5" x 3/8"	2 No. 6" x 6" x 3/8"

L2-L3	14” x 5/16”	2 No. 17” x 5/16”	2 No. 2.75” x 2.75” x 3/8”	2 No. 6” x 4” x 3/8”
L3-L4	14” x 5/16”	2 No. 17” x 5/16”	2 No. 2.75” x 2.75” x 3/8”	2 No. 6” x 4” x 3/8”
L4-L5	14” x 5/16”	2 No. 17” x 5/16”	2 No. 2.75” x 2.75” x 3/8”	2 No. 6” x 4” x 3/8”
L5-L6	14” x 5/16”	2 No. 17” x 5/16”	2 No. 2.75” x 2.75” x 3/8”	2 No. 6” x 4” x 3/8”

Top chord/Vertical/Diagonal members

All top chords, vertical, and diagonal truss members appear to have 2.25” x 5/16” single lacing bars on both sides of the members. The first vertical member above the arch support (L0-U0) consists of unequal angles, plates on two sides and single lacing on the other two sides. All the other elements consist of two channels with single lacing on either side. The following tables summarise the member sizes, based on the site measurements provided by Lumen, that were used for this assessment. Note that the top chord and the vertical/diagonal members connecting at nodes L1, L4 and L5 were not included in the site investigation such that the member sizes are as originally interpreted from the drawings.

Table 4-2 – Vertical member L0-U0 member sizes used in assessment

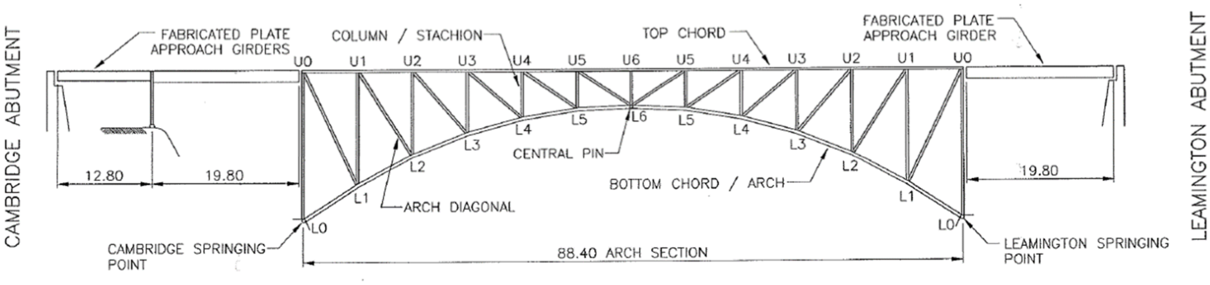
Member	Webs	Angles
L0-U0	2 No. 19” x 5/16”	4 No. 4” x 3” x 5/16”

Table 4-3 – Top chord, vertical, and diagonal member section sizes used in assessment

Member	Side channels
Top chords (typ)	2 No. 10” x 20 lb/ft
L1-U1	2 No. 15” x 33 lb/ft
L2-U2	2 No. 9” x 19 lb/ft
L3-U3	2 No. 9” x 19 lb/ft
L4-U4	2 No. 9” x 20 lb/ft
L5-U5	2 No. 9” x 20 lb/ft
L6-U6	2 No. 9” x 19 lb/ft
U0-L1	2 No. 15” x 33 lb/ft
U1-L2	2 No. 9” x 19 lb/ft
U2-L3	2 No. 9” x 19 lb/ft
U3-L4	2 No. 8” x 13.75 lb/ft
U4-L5	2 No. 8” x 13.75 lb/ft
U5-L6	2 No. 9” x 19 lb/ft

Member diagram

Figure 4-1 – Truss member notation



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4.2 Main Arch Assessment Results

The main member axial capacities determined in accordance with the section properties provided above are summarised in

Table 4-4 below. The compression and tension capacities have been determined in accordance with AASHTO.

The slenderness ratio of the lacing elements used does not comply with AASHTO which specifies a maximum slenderness ratio 140 for single lacing. The lacing slenderness ratio in the main members is up to 193 in the bottom chord members and up to 170 for the top chord and vertical plane members.

AASHTO also requires (refers to AISC Specification for Structural Steel Buildings 2016) that the lacing be designed for a shearing force equal to 2% of the axial strength of the member in addition to any shear forces due to local lateral effects and loads (wind, seismic etc). This shearing effect is not a physically applied load, but a design requirement to ensure that the main element can dependably achieve its' axial capacity without the buckling of any component of the built-up member.

Victoria St bridge lacing does not have sufficient capacity to carry shear equal to 2% of member nominal axial capacity. Therefore, axial loads in the member would have to be proportionally limited to satisfy AASHTO. In the bottom chord this would limit total axial compression to 2000kN.

To check this approach, we also looked at New Zealand code provisions. NZS 3404 also sets a limit on slenderness of lacing at 140. The design 'shearing force' required for lacing specified in NZS 3404 and AS/NZS 5100.6 varies depending on the magnitude of the axial load in the main member and main member slenderness and compression capacity, according to the formula below. This formula generates a lower shearing force than the 2% of axial compression specified by AASHTO (but not less than 1%), which would allow the Victoria St Bridge lacing to sustain a higher axial compression in the main member. This would allow a maximum axial load of approximately 4000kN in the bottom chord prior to exceeding the lacing buckling capacity. This indicates that the lacing is adequate to allow development of the compression capacity in the arch members, when the formula provided in NZS 3404 and AS/NZS 5100.6 is utilised.

$$V^* = \frac{\pi\left(\frac{N_s}{N_c} - 1\right)N^*}{\lambda_\eta} \qquad \text{. . . 10.4.1(1)}$$

$$\geq 0.01N^* \qquad \text{. . . 10.4.1(2)}$$

where

N_s = nominal section capacity of the compression member specified in Clause 10.2.1

N_c = nominal member capacity of the compression member specified in Clause 10.3.3

N^* = design axial force applied to the compression member

λ_η = modified member slenderness

Further checks on the compression capacity of the bottom chord have been carried out assuming that the lacing is fully ineffective, to derive a conservative lower bound estimate. The flexural-torsional buckling capacity was calculated in accordance with AASHTO and found that the compression capacity reduces by approximately 30%, which would not be adequate for the assessment loads. For this case the section was considered as a built-up channel section to replicate the angles and plates forming the three sides of the cross section, ignoring the lacing that is attached to the bottom face. This lower bound conservative estimate has only been carried out as a theoretical assessment and is not considered representative of the actual capacity.

NCHRP Report (333) provides a method to adjust the buckling capacity of the main members by calculating a reduction coefficient 'Ql' to account for corrosion of the lacing. This reduction factor has been determined for the actual (uncorroded but non-compliant) lacing size and used in the determination of the axial compression capacity. This results in only a small reduction in member axial capacity. We note however that this coefficient assumes that the lacing complies to the slenderness limits and does not specifically account for the compression buckling strength of the lacing. We therefore do not believe that it would be appropriate to assume that this reduction factor would take precedence over the shearing load discussed previously.

$$Q_l = \left(1 + \frac{4.8\ I}{A\ L^2 \cos^2\theta \sin\theta}\right)^{-1}$$

Table 4-4 – ULS Case 1 (Cars)

Member	Identification	Capacity (C)		Critical Demands (D)		Critical D/C ratio
		Compression Capacity (kN)	Tension Capacity (kN)	Compression (kN)	Tension (kN)	
Top chord members	All	1096	1273	115	182	0.14 (2)
Bottom chord members	**LO-L1**	3308 (1)	3572	2802	n/a	0.85(1)
	L1-L2	2801 (1)	2993	2640	n/a	0.94(1)
	L2-L3	2560 (1)	2724	2545	n/a	0.99(1)

	L3-L4	2572 (1)	2724	2408	n/a	0.94(1)
	L4-L5	2576 (1)	2724	2320	n/a	0.90(1)
	L5-L6	2507 (1)	2646	2257	n/a	0.90(1)
Vertical Members	**U0-L0**	1297	2115	248	n/a	0.19
	U1-L1	1360	2026	322	n/a	0.24 (2)
	U2-L2	612	1143	276	n/a	0.45
	U3-L3	808	1143	333	n/a	0.41
	U4-L4	990	1225	277	n/a	0.28 (2)
	U5-L5	1063	1225	327	n/a	0.31 (2)
	U6-L6	1010	1143	205	n/a	0.20
Diagonal members	**U0-L1**	1265	2026	188	n/a	0.15 (2)
	U1-L2	485	1143	110	40	0.23
	U2-L3	641	1143	74	44	0.12
	U3-L4	498	847	n/a	121	0.14 (2)
	U4-L5	543	847	44	85	0.10 (2)
	U5-L6	800	1143	139	96	0.17

Table 4-5 – ULS Case 2 (One lane of 0.85HN)

Member	**Identification**	**Capacity (C)**		**Critical Demands (D)**		**Critical D/C ratio**
		Compression Capacity (kN)	**Tension Capacity (kN)**	**Compression (kN)**	**Tension (kN)**	
Top chord members	All	1096	1273	333	389	0.31 (2)
Bottom chord members	**LO-L1**	3308 (1)	3572	3416	n/a	1.03(1)
	L1-L2	2801 (1)	2993	3272	n/a	1.17(1)
	L2-L3	2560 (1)	2724	3207	n/a	1.25(1)
	L3-L4	2572 (1)	2724	3051	n/a	1.19(1)
	L4-L5	2576 (1)	2724	2985	n/a	1.16(1)
	L5-L6	2507 (1)	2646	2871	n/a	1.15(1)
Vertical Members	**U0-L0**	1297	2115	491	98	0.38
	U1-L1	1360	2026	570	n/a	0.42 (2)
	U2-L2	612	1143	496	n/a	0.81
	U3-L3	808	1143	526	n/a	0.65
	U4-L4	990	1225	440	n/a	0.44 (2)
	U5-L5	1063	1225	493	n/a	0.46 (2)
	U6-L6	1010	1143	362	n/a	0.36
Diagonal members	**U0-L1**	1265	2026	358	198	0.28 (2)
	U1-L2	485	1143	243	232	0.50
	U2-L3	641	1143	177	219	0.28
	U3-L4	498	847	44	278	0.33 (2)

U4-L5	543	847	190	229	0.35 (2)
U5-L6	800	1143	364	293	0.46

Notes:

1. The compression capacity provided in the tables above for the bottom arch members assume that the lacing will not limit the development of the full member capacity based on the NZS 3404 and AS/NZS 5100.6 lacing shearing requirements.
2. The capacity of the top chord and the vertical/diagonal members connecting at nodes L1, L4 and L5 is based on the member sizes interpreted from the drawings, as these elements were not verified with site measurement in the June 2021 investigation by Lumen. However this is not considered critical given the relatively low demands in these members.

Due to the negligible secondary bending demands in the members, only axial demand to capacity ratios are compared above. Demand / capacity ratios exceeding 1.0, indicating inadequate strength for the demands, are highlighted in red.

5 Gusset Connection Assessment

5.1 Selection of Gusset Connection

One gusset connection has been assessed. This was chosen for assessment based on the below rationale.

The rationale for the selected connection is that it has the highest combined load transferred through the gussets from the inclined and axial members into the bottom chord, indicating that these gussets are likely to have the highest shear stresses of any on the structure. From the analysis model, arch gusset connection L3 has the highest difference in shear and axial demand between the members. The plate thickness of the gusset is 5/16'' (7.94mm) thick and gusset connection L4 is also the same thickness plate but lower number of bolts and loads, therefore L3 would likely be more critical than L4.

However, it should be noted that each connection is different and therefore it is possible that this is not the most highly stressed connection.

It is noted that the 1996 assessment report indicated that the arch members between L2 and L4 were the critical members for determining load carrying capacity, and this also contributed to selection of gusset L3 for assessment,

5.2 Connection Geometry and Material Properties

Connection details (geometry, plate thickness, steel grade, bolt arrangement etc) were extracted from scans of the original drawings and material strengths from original construction records.

The gusset plate is 5/16'' thick for the selected gusset. Other gussets range between 5/16 and ½" thick.

5.3 Gusset Capacity Assessment Standards

Assessment was carried out in accordance with AASHTO Manual for Bridge Evaluation, 2nd Edition, 2011, including 2016 Interim Revision.

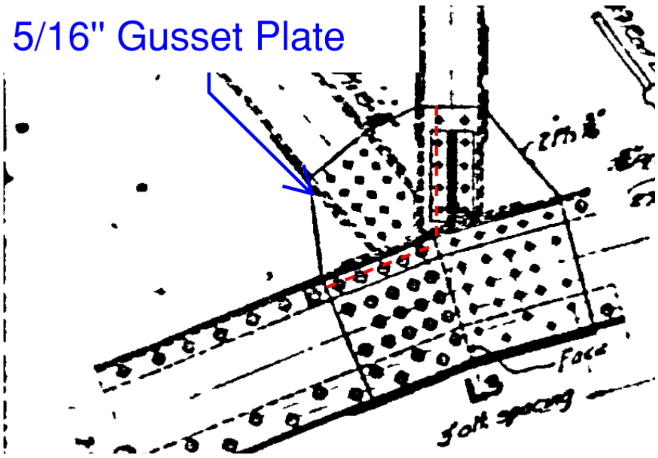
5.4 Assessment Method

5.4.1 Partial Shear Plane Check (PSP)

A simple but conservative check was used to check this gusset plate. The check chosen was the Partial Shear Plane check.

Figure 1 below shows the location of the partial shear planes assessed on the gusset connection. This check considers gross section shear yielding along the vertical and horizontal "partial" shear planes adjacent to the compression member as shown as red dotted lines the figure below.

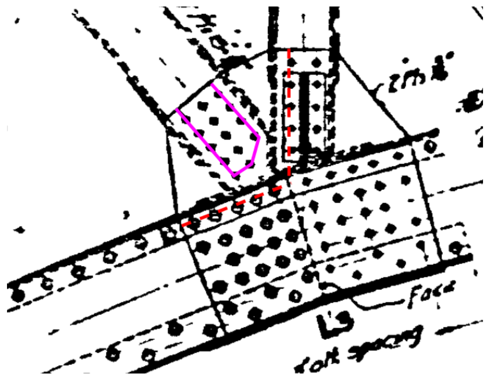
Figure 5-1 – Vertical and horizontal Partial Shear Planes for L3 Gusset



5.4.2 Gusset Plate Tensile Resistance

The Gusset Plate Tensile Capacity check has been undertaken based on block shear failure as indicated along the magenta coloured line in the figure below.

Figure 5-2 – Indicative shear block used for assessment



5.4.3 Arch Splice Capacity

The truss arch bottom chord members are spliced together within the gusset connections. The gusset splice connection consists of two side gusset plates, plus top and bottom splice plates. These are assumed to transfer the full compression load between each of the main arch members. This may be a conservative assumption as the ends of compression members may be machined to a tight tolerance and installed to bear directly onto one-another, in which case AASHTO MBE allows a portion of the compression to be considered to transmit directly, so reducing demands on the gussets. Total compression capacity of the splice was checked. Capacity reduction factor considered for this check is 0.85

5.4.4 Gusset assessment results

Table 5-1 – D/C ratios for Gusset checks

Gusset Check	Case 1 - Cars (D/C)	Case 2 - 0.85HN (D/C)
Partial Shear Plane Check	0.69	0.92
Plate compression check	0.58	0.93

Gusset Tensile Capacity Check	<0.3	<0.4
Arch Splice Capacity	<0.8	<0.8

A site inspection was undertaken to confirm that the arch members bear on to each other at the splice location.

6 Central Pin Assessment

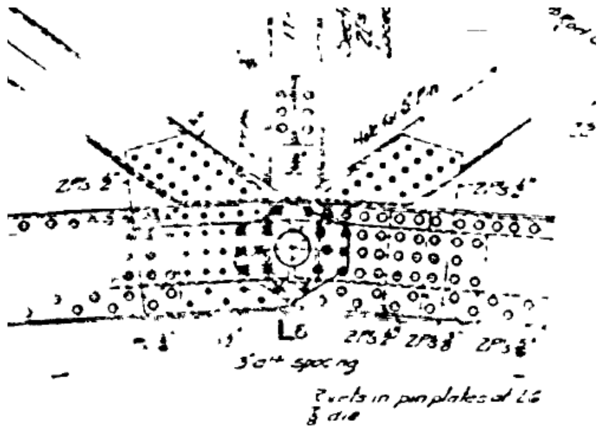
6.1 Background to assessment

The bridge has a central 6" (152mm) diameter pin at mid length of the main arch span (on each side of the bridge) as indicated in Figure 6-2. The pin transfers axial loads between the main arch members, and the vertical and diagonal members that also connect to the pin.

There is significant corrosion in the plates that connect to the pin while the pin itself appears to be in relatively good condition. The 1996 assessment report (BBO) calculated the capacity of the pin connection to be 2125kN. We understand this was based on a yield stress of 200MPa, with an allowance for deterioration in the bearing plates of 25%. The central pin joint was not considered to be the most critical component of the bridge and was reported by BBO to provide 74% of the live load demand (0.85HN), increasing to 158% of the live load demand if the deck sliding joint is assumed to have seized up.

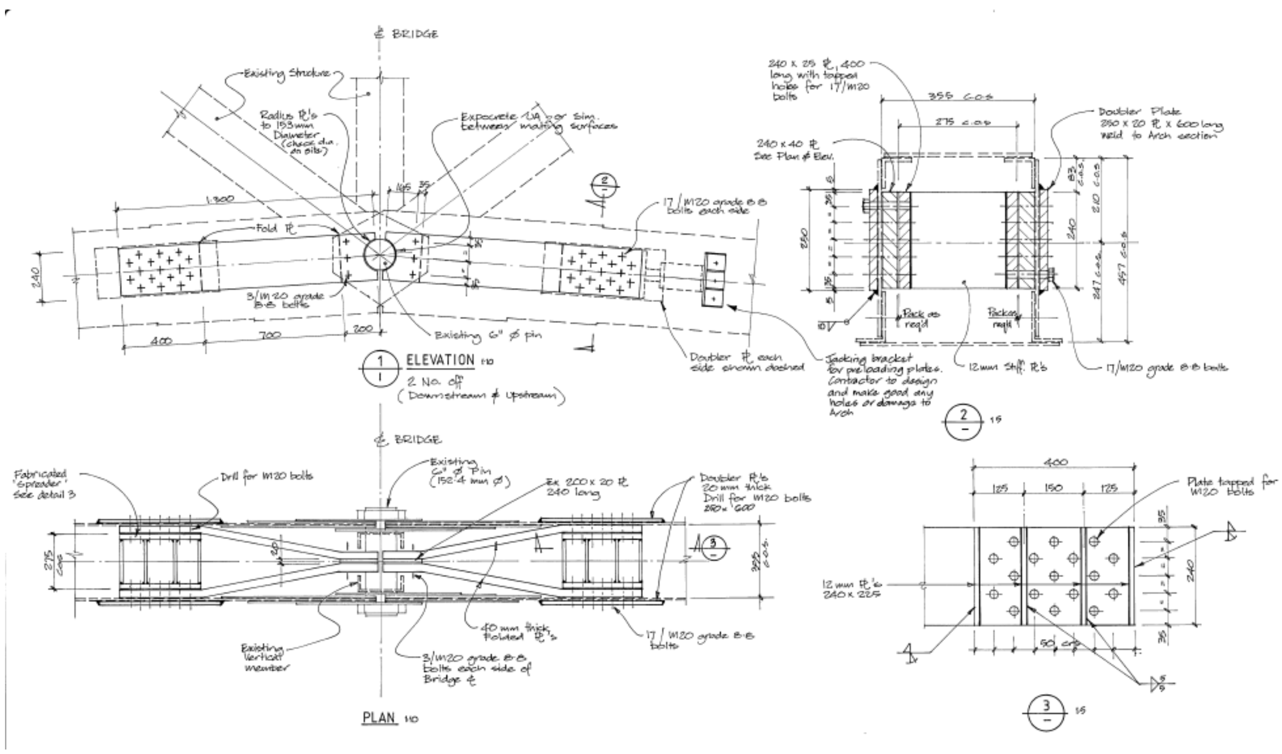
Figure 6-1 – Central Pin (viewed from below) Plate Corrosion

Figure 6-2 – Central Pin Original Construction Details



In 1998 (approximately) additional plates were installed at the central pin joints to improve the capacity of the connection, although it is not clear if these plates were designed to take the full design load or a portion of it.

Figure 6-3 – Central Pin Upgrade Details (1998)



In a subsequent assessment for widening the footpaths in 2012, BBO reassessed the critical arch members capacity and demand, but did not revisit the pins, presumably as they were not considered to be a critical component.

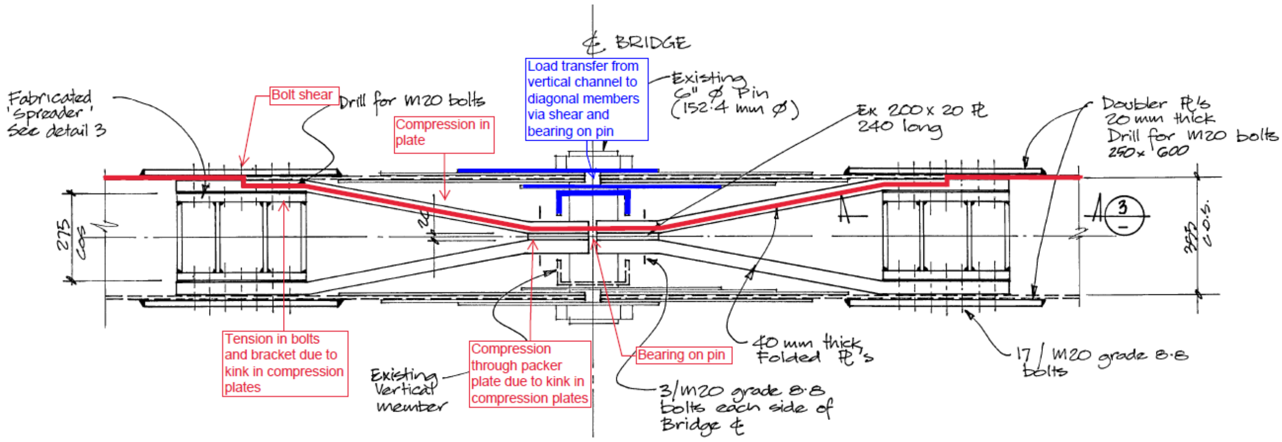
6.2 Current Pin Assessment Results

We have calculated the capacity of central pin and the additional plates installed circa 1998.

The 1998 pin upgrade plates transfer compression between the main arch members either side of the joint. The load path through these plates is indicated in red in Figure 6-4 below. If the original load path is compromised due to corrosion the load will divert to the upgrade plates. The load path through the additional upgrade plates has been determined to have a capacity of 2700kN. This is based on an assumed yield strength of 200MPa for the original arch truss elements and 250MPa for the upgrade plates. The assessment considered pin shear, pin bearing, plate axial section capacity, plate axial member capacity and combined bolt shear and tension. No allowance has been made for deterioration (wear or corrosion) in the upgrade plates or original pin. The limiting load path of the upgrade detail is bearing stress between the pin and plates, but we note that the combined bolt shear and tension is not far behind. The upgrade plates have been assessed to have a capacity similar to the original plates.

Compression in the vertical members above the pin transfers to the diagonal members and arch chord members via the original bearing plates. The load path through these plates is indicated in blue in Figure 6-4 below. If the load path via the original plates is compromised by corrosion in the plates the load could transfer to the arch chord via the upgrade plates, however, this will result in bending and shear in the pin due to the offset between the plates. The pin bending and shear capacity is calculated to be adequate for the ULS Case 1 (cars) demands in the vertical and diagonal members, should these loads need to transfer to the upgrade plates. Note however that if the axial loading in the vertical arch members cannot transfer to the pin due to excessive corrosion in the attachment plates, then the load carrying capacity of the bridge deck structure would be compromised. Hence the integrity of the connection between the vertical channels and the pin needs to be maintained.

Figure 6-4 – Load path for axial loads in arch members(plan view)



7 Summary of structural assessment

It was found that the 1999 central pin strengthening plates can accommodate the demand for the current operating load and reliance on the existing bottom chord bearing plates are not required to bear on the central pin to maintain structural integrity of the bridge.

The structural assessment of the main arch revealed that the section capacities and demands on the structure are lower than previous assessments undertaken. Notwithstanding this, under the current operating case, no strengthening of the bridge is considered to be required, and specific findings relating to the assessment are outlined below.

It was found that:

- The lacing of the main arch members do not meet the slenderness requirement. The lacing strength requirement of AASHTO is not satisfied, but is for NZS3404 and AS/NZS 5100.6 and for the assessment loading this is considered an acceptable standard.
- The arch splice gusset connection was found to pass the assessment standard.
- The critical member for the main arch was member L2-L3, with the remaining bottom chord members only slightly less critical.

- The main arch has sufficient capacity to accommodate the ULS load combination for case 1 loading, which is a continuous stream of two lanes of light vehicles simulated as two lanes of 1.5 tonne axles at 5m centres.
- The main arch does not pass the assessment standard to accommodate the ULS load combination for 0.85HN loading (case 2 in this report). It is noted that these loads are not allowed on the bridge.
- The strengthening plates for the pin are adequate to accommodate the total load for the ULS case 1 (Cars), but not ULS case 2 (0.85HN). They were found to provide an alternative load path to the original bearing plates.

We have assumed the load path between the vertical and inclined members at the pinned connection are in reasonable condition. It is possible that the diagonal and vertical members may be compromised by corrosion, refer to next section for recommendations.

8 Recommendations for future bridge management

We recommend:

- Remedial repair details for the original central pin bearing plates from the bottom chord should be developed to restore reliable full load transfer of the original pin bearing mechanisms, as there is a potential risk if the outer plates deteriorate further the pin assembly may become laterally unstable from eccentric loading. As these are at load bearing interfaces, the repair will not be straight forward.
- Maintenance to the corroded steelwork be undertaken to prevent further sectional loss.
- For future repainting of the structure, under the assessed bridge operating case, the main arch has limited to no residual axial load carrying capacity to accommodate additional load due to scaffolding and live load on the scaffolding. Vehicle or pedestrian access to the bridge will therefore need to be restricted to accommodate the temporary works loads on the bridge during maintenance works. This will need to be reviewed prior to committing to repainting to confirm scaffolding loads can be supported by the structure.

These recommendations are made in context of the current restricted loading regime, and assumes existing controls remain effective at restricting vehicles exceeding the posted limit from accessing the bridge.

A

Appendix A – Rope Access Inspection Report

Victoria Bridge-Cambridge Steel Components Inspection Report

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Rope Access inspection of specific bridge components.

Job No. CAP18426

June 2021

REVISION HISTORY

REVISION	DATE	DESCRIPTION	PREPARED BY	REVIEWED BY	APPROVED BY
1.0	11 June 2021	Draft	M van Rooyen	M Boardman	M van Rooyen

Client Contact:	Liam Edwards
Client Reference:	5640849
Project Title:	Beca-WDC Victoria Bridge Cambridge RA Inspection

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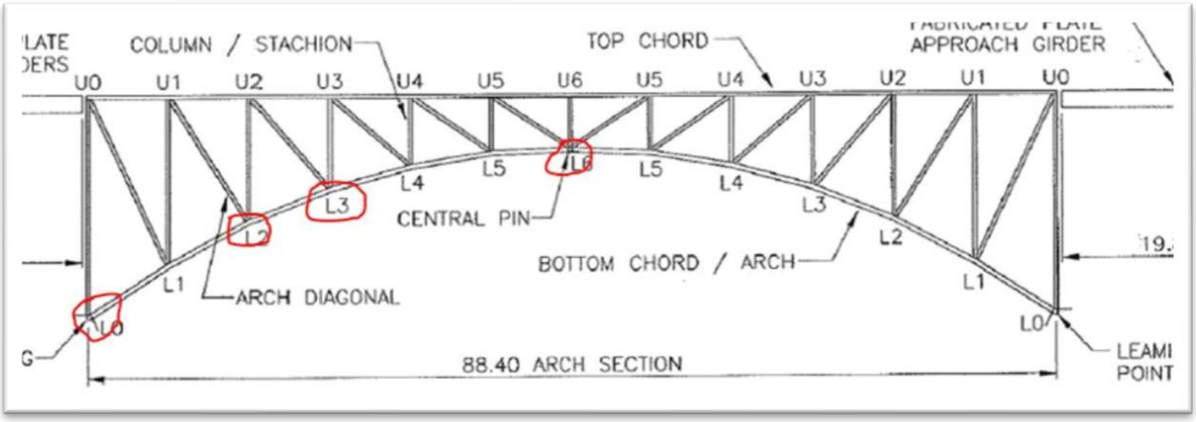
EXECUTIVE SUMMARY

A visual assessment of the predetermined structural elements on the Victoria Bridge, Cambridge was carried out in June 2021 by Lumen. Components that were inspected were at connection points of the bottom arch on L0, L2 (upstream and downstream), L3 (upstream and downstream) and L6/Central Pin (upstream and downstream). Multiple steel member measurements, a visual assessment of corrosion and metal loss as well as other potential defects were inspected and recorded.

Inspection Locations



Measurements of steel components

	Cover Plate width/thickness	Web width/thickness	Top L	Bottom L (v/h/thickness)	Vert/Diag Lacing	Arch Lacing	Gusset Plate mm / thickness	Vertical UC	Diag UC	Vertical Angle / L	Vertical Plate	Spreader Plate	Other
L0	360/9	430/9	90/90/10	150/150/15	320/9	585/9	15	-	-	75/100/9	480/9	-	103/33/15
L2 (L1 side)	360/9	430/9	90/90/10	155/155/10	340/9	540/9	9	225/65/10	225/70/10	-	-	-	-
L2 (L3 side)	-	-	70/70/10	150/100/10	-	540/9	-	-	-	-	-	-	-
L3 (L2 side)	360/9	430/9	70/70/10	150/100/10	340/9	540/9	9	225/65/10	225/70/10	-	-	-	-
L3 (L4 side)	-	-	70/70/10	150/100/10	-	540/9	-	-	-	-	-	-	-
Central Pin	360/9	430/9	60/60/10	150/100/10	340/9	540/9	9	225/65/10	225/70/10	-	-	600/250/20	565/330/10

CORROSION SITES

All corrosion sites that were visually inspected have some degree of metal loss. Most of the sites are made up of either “crevice” corrosion or “exfoliation” and are in areas where multiple steel components join at spliced connections. A combination of fungal growth and joint sealant has caused dirt entrapment and in areas where the design of the bridge does not allow for natural run-off and cleaning to occur. The central pin (upstream and downstream) and the cover plates between L0 and L1 (downstream) are in the worst condition. The corrosion around the central pin does not appear to affect the central pin itself and is only showing signs of coating delamination and minor metal loss where the pin slots through the vertical “U” member. The vertical “U” member is completely exfoliated when viewed from the bottom up on both ends of the bridge.

The ends of both the bottom and top “L” members where the central pin meets the steel arch have progressed to the perforation of the steel. Where the cover plate joins on top of the internal “L” member a combination of “crevice” corrosion and “exfoliation” is occurring. Previously installed joint sealant as well as excessive fungal growth in these areas have caused moisture retention and dirt entrapment which subsequently resulted in corrosion and metal loss of the said area. The edges of the top plate were not able to be measured to obtain accurate measurements for metal loss as most of the long edge of the plate has progressed to a jagged profile due to corrosion.

The top edges of the gusset plates where the plate meets the vertical and diagonal columns have some degree of “crevice” corrosion. These areas have not progressed to severe metal los though. L2 (upstream/internal face) being the worst.

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APPENDIX A - PHOTOS






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