James St Lithgow Underbridge Assessment

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ABSTRACT

The James Street underbridge is a stone masonry arch rail bridge built in 1869. The bridge is west of Lithgow Railway station and carries two tracks as part of the Sydney Trains network. The bridge is of heritage significance as it is the second oldest stone arch railway bridge in NSW still in use and was designed by renowned engineer John Whitton.

Aurecon undertook a load rating assessment with the aid of specialist arch analysis software and with guidance from Australian and international standards and codes. The findings of the assessment necessitated the detailed design of arch crack repairs and lateral strengthening to enable provide adequate capacity for current freight loading and further extend the service life of this historic structure.

A number of challenges encountered on the historic James St underbridge from the initial investigation and assessment are presented within this paper, which may be of consideration on other unique structures. The behaviour of stone masonry arch structures, and the significance of defects such as arch cracking on the load rating and refurbishment design are some of the important features.

1 INTRODUCTION

The maintenance of existing bridge structures is an instrinsic part of asset management. Specifically, on the Sydney rail network, standardised processes and guidelines are followed for the assessment and repair in order to ensure a reliable and safe service outcome.

Aurecon was commissioned by Sydney Trains to undertake a site investigation, condition and load rating assessment for the existing underbridge at James Street in Lithgow in 2016.

The James St underbridge was designed by the preeminent engineer John Whitton, considered to be the Father of NSW railways. Constructed in 1869, it carries two tracks of the Western Main Line and is a critical piece of infrastructure in the Blue Mountains.

The masonry arch bridge is 27m long overall and 8.2m wide, consisting of three sandstone semi-circle arch barrels each of 6m span (Figure 1). The arch barrel has a clear height of approximately 4.2m from the road surface to the central key stone. The two tracks are supported on ballast fill with 7.2m clearance between the walls.

Figure 1 Elevation of the underbridge arches

2 SITE INVESTIGATION

Three separate investigations were undertaken on the bridge; the initial detailed examination, a follow-up investigation into the bridge deck composition and dynamic response monitoring.

The initial investigation was undertaken in accordance with ASA documents TMC 301 Structures Examination and TMC 305 Structures Assessment.

No original drawings were available of this historic bridge structure. This initial investigation measured the arch layout, dimensions of sandstone blocks or “voussoirs” and grout thickness to undertake the load rating. Potholing of the ballast deck during the possession identified the fill behind the arch. At 900mm below ballast level, a concrete slab was encountered. This unanticipated finding influenced the assumptions on the arch behaviour and the method of analysis.

The significant defects identified were:

- Longitudinal cracks in the arch immediately behind the spandrel wall and running through stones
- Longitudinal cracks observed at the centre of the intrados (inner face of arch)
- Sign of water seepage along joints and cracks in the arches, notably at the centre of arch and at spandrel wall
- Smaller horizontal cracks observed at the top of the spandrel wall
- Mortar missing in between stones in spandrel wall observed from the corridor
- General weathering of the masonry with surface defects

The second investigation was undertaken following the load rating and condition assessment, using Ground Penetrating Radar (GPR) scanning across the ballast in the rail corridor to determine the extent and thickness of the concrete slab below the ballast and determine the thickness of the voussoirs. An estimate of the compressive strength of the sandstone was determined using rebound hammer.

The outcomes of this investigation confirmed the concrete slab continued across the three spans, but due to high moisture levels and ballast condition, the GPR results were not definitive in terms of the extent of the slab and materials below, upon which the load rating assessment was to be based.

This finding necessitated an approach where assumptions on the fill material and arch thickness for load rating were based on what was immediately visible and reasonably determined from the scanning.

3 CONDITION ASSESSMENT AND LOAD RATING

3.1 Condition

The inspection findings concluded that for its age and years in service, the bridge was in a moderate condition overall. The key defects affecting the structural integrity were the longitudinal arch barrel cracks at the arch centre and immediately behind the spandrel wall.

The spandrel cracks indicated that lateral pressures were significant, likely generated from the train loads being in very close proximity to the wall which are non-compliant to the current ASA transit space guidelines.

The central crack is a result of stresses most likely caused by a combination of bi-directional train loading, possible settlement and inefficacy of the tie bars to provide lateral support. The arch behaviour remains essentially the same.

The spandrel defects are not typically associated with a reduction in arch load carrying capacity, but affect the stability of the ballast track. The spandrel walls provide support to the fill upon which the load is distributed to the arch and tie rods provide the additional support to the walls.

Historical photos show that the tie rods could have been installed by at least 1883. The presence of the tie rods at this early age of the structure life, coupled with the site observation of water seepage through arch cracks supports the assumption that the tie rods had corroded and been rendered ineffective, with the longitudinal cracks generated afterwards. The tie rod plates were also observed to be disconnected to the bridge walls.

The initial assessment conservatively assumed that the arch was performing beyond its serviceability limit state and should be monitored for further movement. However, by comparing photos taken in 2006 & 2016, the longitudinal cracks did not show signs of growth and may have remained stable for a number of years. The risk of progressive differential settlement was then considered small and the spandrel wall behaviour took priority.

3.2 Load rating

3.2.1 Methodology

The load rating of arches is typically undertaken in accordance with AS5100 Bridge Design and ESC 310 Underbridges. Sydney Trains confirmed that the bridge capacity did not need to be rated for standard 300LA but the MF freight loading.

Methods to assess arch structures are typically found in international standards; notably the British Assessment of Highway Bridges documents BD 21/01 and BA16/97.

A common, simplified method of arch assessment is the MEXE method. This was not appropriate for the bridge due to the following constraints in its application:

- Multiple span bridges are not covered by MEXE Method
- The 2.5m rise is greater than Span/4, which is not covered by the MEXE Method
- Fill material above the crown (1300mm) is greater than the maximum of 1050mm
- The longitudinal concrete slab may be directly supported by the arch barrel extrados and can have a significant stiffening effect. Relieving arches were typically designed to keep the line of thrust close to the arch shape. MEXE Method is also not applicable for this feature.

A specialised arch analysis software Archie-M was then used to undertake the analysis. However, Archie-M does not consider spandrel walls in the arch analysis but determines the line of thrust under moving loads. In this case, the analysis would only consider the arch behaviour and not the three-dimensional arch structure. There was no facility to incorporate a concrete relieving slab; manual calculations distributed the train loads as an approximation of the slab behaviour.

Since the Archie-M analysis does not provide an arch “capacity” value, an alternative process to determine the load factor was nominated. The live load factor software input was increased from the code requirement until arch failure was predicted; the ratio of revised load factor over the code value was the LRF.

3.2.2 Condition factors used in load rating

In order to determine an “As Is” rating, there needs to be some compensation made for the structural condition. This is usually a straightforward task when, for example, structural elements are formed of steel and a reduced cross section can be readily measured and lower capacity calculated. Arch behaviour still remains extremely complex to model accurately despite its use over the millennia. This calls for calibrated, finite element models where all material properties are known.

Correlation of the defects needed to be made using a reduction factor since the spandrel arch cracks could not be modelled within the software. The British assessment guidelines and other international references provided this approach.

A condition factor was adopted to reduce the capacity based on the severity and class of defect as defined within BA16/97. Clause 3.17 states: ‘The factor for the condition of the bridge depends much more on an objective assessment of the importance of the various cracks and deformations which may be present and how far they may be counter-balanced by indications of good material and workmanship.'

The factor of 0.7 was chosen for the longitudinal cracks, which did not appear to be progressively deteriorating and no settlement or arch movement issues were evident (no lateral cracks, no diagonal cracks). There was also no evidence of weakness at the base of the piers.

In this case, the noted defects were considered non-critical to the load carrying capacity of the arch, and after only minor repairs, the condition factor could be increased (Ref Cl 3.18 BA16/97).

The quality of the sandstone and workmanship was demonstrated by the uniform sandstone blocks and continuing service of the structure. The defects present at James St were considered relatively minor where the expected repairs would involve bridging cracks and maintaining the arch behaviour. A revised condition factor of 0.85 was considered suitable following completion of the repairs.

3.2.3 Load rating outcome

A load rating factor (LRF) of 1.19 was calculated for 'As New' and 0.83 for 'As Is' for the MF train load. The outcome of the Archie-M analysis indicated the internal piers were the critical element for failure when adopting the condition factor of 0.7; this factor being objectively determined by observation on site and correlation with standard practices. However, the analysis result was not supported by any evidence of deformation and/or cracking within the piers on site.

A sensitivity analysis was undertaken to determine if the assumed sandstone strength or grout thickness significantly affected the outcome. A compressive strength of 10 and 15 MPa was compared with only minimal improvement. A load rating greater than 1 could be achieved for MF loads provided that the train speed was restricted to 40 km/h. This was the speed restriction already in place.

Sensitivity Analysis		**Load Rating Factors**			
Compressive Strength (MPa)		**As New**	**As Is**		
	Rating Vehicle LRV	**Load Rating**	**Load Rating for 100% DLA**	**Load Rating for 50% DLA**	**Reduced L_{RV} for RF=1**
10	**300LA**	0.80	**0.56**	**0.66**	**169**
	MF	1.19	**0.83**	**1.03**	NA
15	**300LA**	0.80	**0.56**	**0.66**	**169**
	MF	1.24	**0.86**	**1.07**	NA

Table 1 Load rating factors

The analysis did not provide conclusive results on the load carrying capacity of the structure. Whilst the capacity of the arch itself was adequate for the MF loading (1.27) the effect of the spandrel walls on the overall stability could not be easily quantified and the thus remained a stability risk.

4 FEA ANALYSIS AND MONITORING

As a result of the load rating results, further investigation was nominated to determine and predict structural behaviour, through monitoring and dynamic response measurement. This level of investigation was warranted due to the significance of the bridge for critical rail services and its heritage value.

This assessment was undertaken by Mainmark to see how the observed defects affected the structural behaviour in service and whether any alleviation of the existing speed restrictions could be made. This model could also be used for future assessment and monitoring purposes as a “base line” model. A load rating can be produced once actual train loads are verified for correlation with the FEA model.

The results of the analysis correlated well with the measurements taken on site. The outcomes of the assessment indicated that the spandrel walls, whether modelled with the crack defects or without, did not appear to have a significant impact on the arch behaviour and its capacity. The longitudinal cracks did not permit optimal load distribution through the full width of arch and there was a significant difference in structural movement between the two sides of the structure, with maximum displacements measured in the structure when directly loaded by the train. Providing additional lateral support would therefore improve behaviour across the bridge. These results therefore correlated well with the recommended repairs. The dynamic response of the piers indicated these elements were relatively stable, commensurate with the lack of defects observed on site.

The Archie M analysis indicating criticality of pier capacity was further discounted as a result of the FEA model and dynamic response outputs. The FEA analysis did support the Archie M analysis for the arch barrel performs well under the current loads in each analysis.

5 REPAIR DESIGN

The British guidelines outline the traditional means of repair for cracking within the arch barrel is by tying both walls together with rods and using large spreader plates on the outside of the bridge. This had formed the original bridge design and supported the undertaking of repairs to spandrel walls.

To improve the overall condition and thus achieve the MF load capacity, the following repairs were nominated as part of the concept design:

- Replacement of tie bars to provide spandrel wall support
- Cracked voussoir stitching and repointing
- Stone surface repairs
- Free draining deck slab providing track lateral support

To improve maintenance safety, new handrail fixings were also nominated.

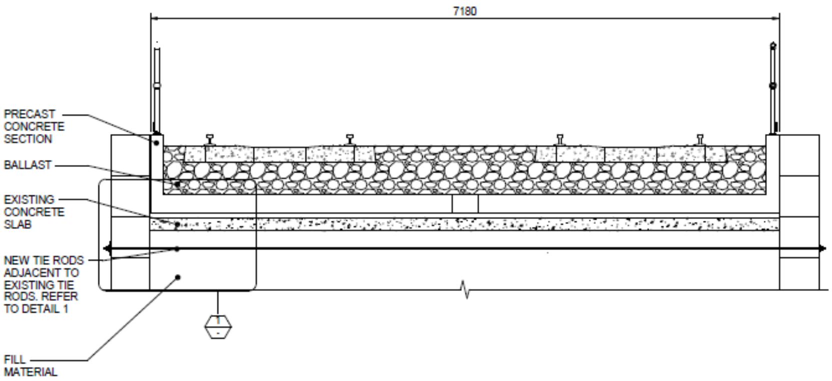

Figure 2 Section of bridge showing repairs

A critical part of the design was complying with the stringent requirements of a heritage listed structure. Features such as avoiding use of electric saws or pneumatic hammers, repointing mortar to be the same or a similar composition, colour, texture, and strength as original, and the visual impact of modifications being controlled by NSW Heritage guidelines.

Time constraints are a common feature of working on the railway. In this case, the repair design took the following into consideration:

- Limited weekend possession windows of 48 hours
- Multiple repair works being undertaken concurrently to be able to complete the whole work within the given time limits

- Repair work on arch intrados and spandrel walls requiring road traffic management
- Solution to be flexible enough to respond to unknown constraints on site

The final design is still subject to further revisions following additional detailed site investigations on the deck structure.

6 CONCLUSION

This three span arch structure required a number of approaches within the investigation and analysis phase in order to determine a realistic assessment result and remedial solution.

A number of different investigations allowed a more comprehensive understanding of the bridge structural form and behaviour. The difficulty in load rating an arch structure required adoption of industry best practice to formulate an appropriate assessment and repair.

Whilst the initial assessment indicated a low load rating with the piers forming the critical section, this was based on a theoretical method which did not fully correlate with site observations. The longitudinal cracks observed on site could not be directly correlated with the load rating but were known to have an effect on stability.

An empirical approach had to be taken to obtain realistic results; the additional site investigations and dynamic monitoring provided additional information that supported the need for repairs to the arch barrel using tie rods and stitching of cracks. The modelling also gave added surety to the initial analysis in confirming the arch was able to carry the designated loading with movements within code limits.

The repairs are predicted to improve the load distribution and overall arch behaviour, supported by industry best practice. Further investigations are also anticipating refinement of design.

Thus the benefit of the FEA analysis was in correlating site measurements on arch movements with a model that could be further extrapolated for future load rating. The modelling of discontinuities for the cracks allowed for a reasonably accurate model that would be a useful tool in reviewing the dynamic behaviour of the arches following the future installation of nominated repairs. This assessment also confirmed the initial analysis findings.

7 REFERENCES

- AS 5100-2017 Bridge Design
- AS ISO 13822 Basis for design of structures – Assessment of existing structures
- ESC 310 Underbridges
- TMC 302 Structures Repair
- TMC 305 Structures Examination
- Vol 3 Section 4 Part 3 BD 21/01- The Assessment of Highway Bridges and Structures
- Vol 3 Section 4 Part 3 BA 16/97- The Assessment of Highway Bridges and Structures
- Safety of Historical Arch Bridges - Dirk Proske & Pieter van Gelder
- CIRIA C656 Masonry Arch Bridges: Condition appraisal and remedial treatment solutions

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9 AUTHOR BIOGRAPHY

Marcia Prelog is an Associate with Aurecon. Her extensive experience in the inspection and assessment of bridge structures in Australia and overseas spans over 18 years in the Bridges discipline with a particular interest in providing innovative solutions to existing bridge assets. Her experience encompasses steel, concrete, composite, masonry and timber structures.