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Load rating and fatigue life assessment of old steel railway bridges; a case study

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ABSTRACT: This study focuses on the methodology and outcomes of a comprehensive engineering investigation on one of the most important railway bridges in the Victorian state of Australia. This investigation aimed to provide an in-depth evaluation of bridge condition, applicable load and speed restrictions and an estimate of service life by means of fatigue assessment.

The original structure built in late 1800's consists of 45 spans of riveted steel superstructure supported on masonry piers and abutments. The investigation consisted of a detailed inspection, modelling and fatigue assessment. Modelling and fatigue assessment was limited to 16 spans of the superstructure in which the original construction was maintained.

The load rating of the bridge revealed the old viaduct fails under current Australian heavy railway design load. However, as the heavy haul is diverted over a new viaduct, the old structure can service passenger trains with no restriction for an extended period of time.

1 INTRODUCTION

A detailed engineering investigation was undertaken by Sterling Group on an old riveted steel railway viaduct in order to estimate its remaining service life and determine whether any operational restrictions *i.e.* load or speed limit due to the structural condition must be enforced.

The viaduct located in Melbourne was originally built in late 1800's and duplicated in the early 1900s to accommodate increasing amount of traffic. It comprises 45 spans in total forming the main link between the eastern and western parts of the Melbourne and Victorian rail network.

This investigation included a comprehensive detailed inspection of the entire structure, structural load assessment with a review of the existing operating conditions and a cost vs. benefit analysis of the proposed works for the original spans, 30 – 45. The cost vs. benefit analysis is not presented in this paper.

For the structural investigation, Australian Bridge Standards, AS5100 and some other relevant railway standards and references were used. The current Australian Standard requires new bridges to be designed for a so called 300LA railway load. This load is much heavier than the load old bridges, such as this bridge, were designed for. Therefore it was anticipated that the bridge will not satisfy this loading.

As the bridge already had operational restrictions limiting the passage of heavy haul, more realistic loading patterns which the bridge is or likely to be subjected to were selected for structural analysis. The bridge has a permanent speed restriction of 25Km/h due to the track geometry which was considered in the analysis as well.

The bridge was thoroughly inspected to collect all data required for the As-Is analysis and design check. All important defects were considered in modelling and the results are believed to be a reasonably accurate representation of the condition of the structure under the given loads and applied restriction. The results from the structural modelling and analyses under various loading conditions were used to identify safe operational speed for passenger or freight trains.

The investigation was completed by undertaking a comprehensive fatigue assessment of the structure to estimate its remaining service life and determine an effective monitoring system once the theoretical fatigue life is reached.

Despite its age in excess of 100 years, and with the exception of a few deficiencies that require minor strengthening or remedial works, the structure showed satisfactory performance under the given loads and applied speed restriction. It was found that the structure can stay in service for an extended period of time with proper maintenance and some remedial works.

2 STRUCTURE DESCRIPTION

The structure consists of two independent bridges, each carrying two tracks on shared piers and abutments. Each bridge comprises two U-Frames with a common girder at the middle (central girders) and one girder each at the edge. The deck or horizontal member of the U-Frames is a troughing deck with infill and overlaying concrete structurally connected to steel girders.

The girders are of riveted plate construction comprising of a single web plate joined by additional flange plates in riveted connection with 4 No. equal angle sections. The girders are ultimately supported on sub-structural steel crossheads and masonry piers. The bridge girders throughout spans 30 – 45 consist of both cantilever and suspended girder types in succession of each other and riveted in connection. This layout gives rise for the girders to behave in a semi-continuous pattern as the loads are transferred throughout all spans. Each span, with the exception of span 45 consists of 6 No. girders, 3 for each bridge respectively causing independence in load bearing between the two bridges. Span 45 consists of 5 No. riveted girders all simply supported. The northern bridge span 45 decking is supported by only two girders as there is no central girder.

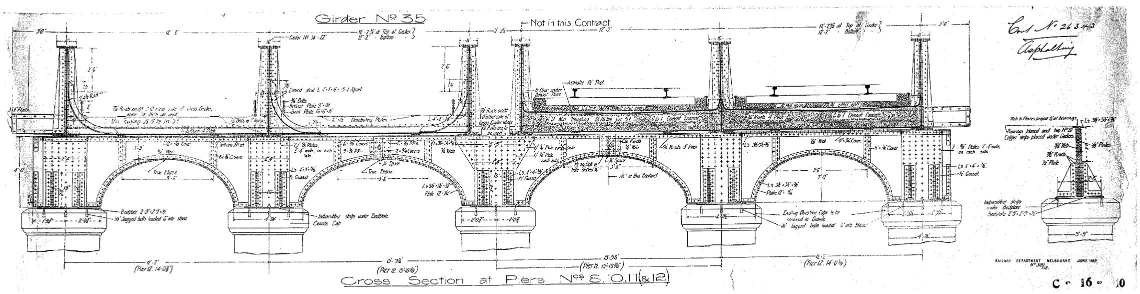

Figure 1: Typical cross section of the superstructure

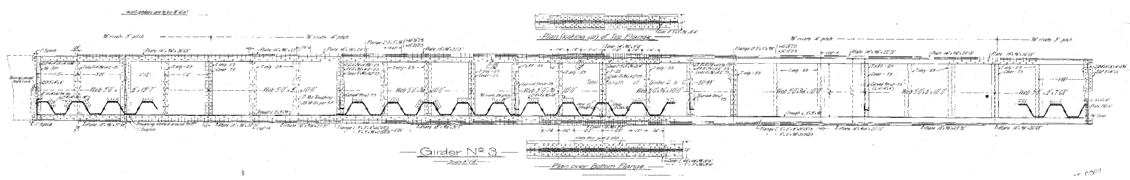

Figure 2: Typical longitudinal view of girders and troughing deck

The U-Frame system provides significant lateral restraint to the compressive flange of the girders and increasing load bearing capacity of these members. Furthermore, curved struts were also installed to further restrain the girders and ultimately enhance the capacity to reach the full bending capacity. Figures 1 and 2 show some details of the structure.

3 INVESTIGATION SCOPE

3.1 Scope of the investigation

The scope of this project was as follows:

a) Undertake detailed visual inspection;

b) Undertake a structural assessment of the viaduct superstructure for spans 30-45 under the following loads;

- 300 LA Design Vehicle per AS5100
- G-Class Loco (×2) and RCNF Steel Wagons (×10) at 21 axle load
- ARTC RAS 210 Loco (×2) and Wagons (×10) at 23t axle load

c) Undertake an analysis and design check on the viaduct for both As-Built and As-Is condition states using measurements taken from the detailed inspection.

d) Produce a fatigue analysis to further determine the viaduct's current condition and remaining service life.

e) Provide a full report on the outcomes of the investigation.

4 INVESTIGATION METHOD AND RESULTS

The methodology used in this investigation consisted of three distinct parts as explained below.

4.1 Detailed inspection and fatigue testing

Detailed inspection was undertaken by using appropriate access equipment to access all important parts of the structure in order to record all defects that may alter structural capacity of the affected members. During the inspection, the extent of each defect was identified and measured as far as practically possible. The recorded defects were used in As-Is modelling of the structure.

During the inspection a limited fatigue test, by means of Magnetic Particle Testing, was undertaken on some critical girders with exposed surface. The inspection report states no fatigue test could be detected on the tested areas.

4.2 Modeling and analysis

Modelling and analysis was carried out by using SpaceGass (2011), a specialized software. The analysis used linear static analysis in which the structure was modelled under predefined and customized moving loads. The mentioned live loads with relevant dynamic effects in accordance with the imposed operational speed restriction were applied.

4.2.1 Live load

The applied live loads are schematically shown below.

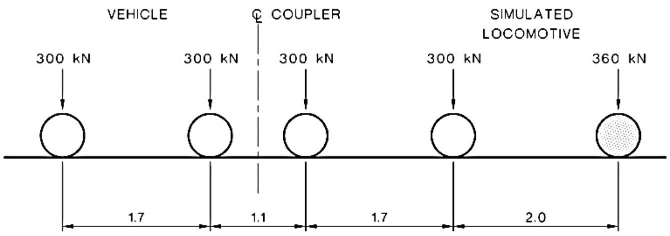

Figure 3. 300LA Load Case Per AS 5100.2 (2004)

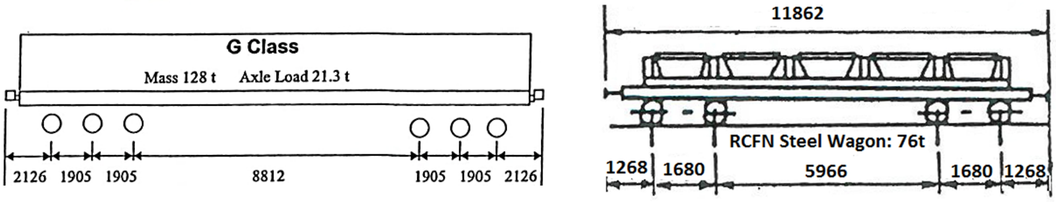

Figure 4. G- Class Loco and RCNF Steel Wagon loading patterns

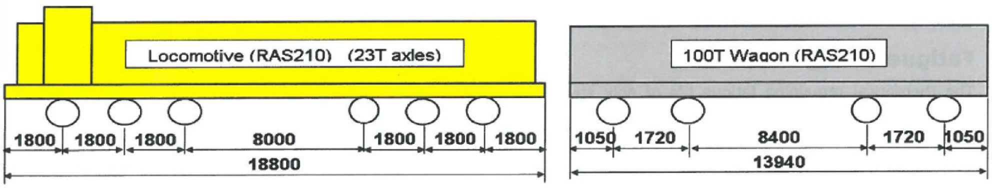

Figure 5. ARTC RAS210 Loco and Wagon loading patterns

The dynamic allowance $\propto$ for ballasted deck of characteristic length of $L_{\propto} > 3.6$ is determined by the following equation.

$$\propto = \frac{2.16}{L_{\propto}^{0.5} - 0.20} - 0.27 \qquad (1)$$

The geometry of the bridge includes a curve for which the centrifugal effect of live load increases the wheel load on one side. This effect was considered through the following equation

$$H_{CF} = \frac{0.0077V^2 A}{r} \qquad (2)$$

in which H_{CF} is centrifugal load, V is design speed (Km/h), A represents axle load (KN) and r represents radius of curve in meter.

4.2.2 Load rating

The rating of a bridge is carried out by comparing the factored live load effects of the nominated rating vehicle with the factored strength of the bridge after subtracting the strength capacities required to meet the factored dead load and superimposed dead load effects and other relevant effects such as differential temperature and settlement etc.

The rating procedure is carried out for all strength checks, *i.e.* bending moment, shear force and the like at all potentially critical sections. The lowest rate factor among assessed members is called the global rating factor and represents the rating factor for the bridge.

The load rating is conducted here in accordance with AS5100.7 (2004). The rating factor (RF) for this specific case can be represented by following equation.

$$RF = \frac{\phi R_u - (\gamma_g S_g^* + \gamma_{gs} S_{gs}^*)}{\gamma_L (1+\alpha)\, W\, S_L^*} \tag{3}$$

in which ϕ is capacity reduction factor, R_u is calculated ultimate capacity, γ_g, γ_{gs} = dead load factors, S^*_g , S^*_{gs} = dead load effects, γ_L = live load factor, S^*_L = live load effects, α = dynamic load allowance and W is accompanying load factor. More details are given in AS5100.7 (2004).

4.2.3 Results

This section presents a summary of obtained results for As-Is condition. The results are based on the yield strength of 170 MPa for rivets and 250MPa for mild steel members.

The structure (girders, deck and expansion joints) were assessed individually for bending moment and shear capacities under all three load cases, each producing its own rating factor, where RF<1 represents a failed member. Rating factors are calculated for structural capacity and dynamic performance for 25km/h permanent operating speeds. The results of this analysis produced a matrix of rating factors for the viaduct, corresponding to imposed loading types.

The performance of the structure was further assessed by taking into account the lateral stability of girders. The curved struts were not considered in this assessment due to their poor condition, therefore the U-Frame system comprises deck and girders only. Despite a significant decrease in effective length of the girders, in many cases full bending capacity could not be achieved due to absence of the curved struts. Tables below present rating factors for As-Is condition.

Table 1. Global rating factor while two tracks of each bridge were simultaneously loaded

Load / Bridge	300LA	G-Class	ARTC-RAS210
North Bridge	**0.66**	**0.96**	**0.75**
South Bridge	**0.73**	>1	>1

Table 2. Global rating factor when only one track of each bridge was loaded

Load / Bridge	300LA	G-Class	ARTC-RAS210
North Bridge	**0.92**	>1	>1
South Bridge	**0.91**		

Table 1 shows the results in which both tracks of each bridge were loaded concurrently. Table 2 on the other hand presents the results when only one track of each bridge was loaded at a given time and the other track had no rail traffic on it. The results show both brides fail under any load configuration of 300LA. This was not an unexpected result as the 300LA loading pattern is a very heavy load and is proposed for design of modern bridges. As mentioned previously, the heavy haul traffic is diverted to a new bridge therefore these old bridges are now to carry passenger or freight trains in which the two remaining load cases represent the heaviest potential load these bridges may experience.

The results confirm that despite the age of the structure it is still in suitably serviceable condition. However, as the North Bridge does not have sufficient capacity in its last span to carry two heavy trains simultaneously, it is required either to divert all heavy loads over the southern bridge or place a restriction to allow one heavy train of any given time on this bridge. Both bridges can carry passenger trains with no operational restriction as the passenger train's crush load is less than 60% of the G-Class load therefore for any load configuration rating factor greater than one will result.

4.3 Remaining fatigue life estimation

The objective of the fatigue assessment is to consider the effects of repeated stress changes imposed on the bridge over 100 years of life and use this information to obtain an approximation for remaining service life.

4.3.1 Rail traffic history

This element of the assessment involved a comprehensive desktop study into the operations of trains across the old viaduct since its opening in 1891. Train timetables from the available records were assessed to estimate the number of trains that would travel across the old viaduct during a standard week.

The earliest available records dated to 1st May 1930. The records were studied in different stages of the viaduct service life and a statistical model was developed to theoretically extend the record over the full span of the viaduct's service life since its opening. Table 3 below summarises the historical as well as the theoretically estimated frequency of train traffic. These numbers were used in the fatigue life estimation and considered as actual load cycles that the structure experienced during its service life.

Table 3 - Summary of Train Frequency History

Period	No. of Years	No. of Tracks / Viaduct ID	Cumulative Train Number	
			Total No. of Trains	No. of Trains Per Track
1891-1914	23	2 / Northern Bridge (old)	2,075,884	1,037,942
1915-1974	59	4 / Northern & Southern Bridges (old)	14,760,921	3,690,230
1975-2012	37	4 / Old Viaduct and 2 / New Viaduct	26,407,242	5,281,448

4.3.2 Fatigue assessment and damage calculation

Fatigue is the failure of a part that is subjected to a repeated load. The failure is caused by the initiation and growth of a crack (or growth from a pre-existing defect) until it reaches a critical size or condition. The aim of a fatigue analysis is to determine if a part will survive the large number of load cycles experienced in its lifetime. Fatigue analysis is used to determine the durability or the fatigue life of a part or member of the structure.

The fatigue impacts on structure are based on two different variables; the maximum stress differences experienced by structural members and the number of cycles. The first one is a result of analyses and the number of stress cycles is determined through the timetable history.

The remaining fatigue life was evaluated based on the procedure detailed in AS5100.6 (2004) and BS 5400 Part 10: 1980. The effective number of load cycles was determined through the historical data and then the Palmgren-Miner summation rule was applied to calculate the accumulative fatigue damage and estimate the remaining service life. The procedure of fatigue design life calculation in accordance with AS5100.6 (2004) is summarised below.

$$D = \sum (n_{iy} / N_i) \quad (4)$$

in which D is fatigue damage that occurs in one year, n_{iy} is the number of cycles of stress range amplitude f_i^* occurring in one year and N_i is the number of stress cycles.

It is well established that the mean stress may have significant influence on fatigue behaviour and therefore on the number of cycles to failure. Generally *S-N* curves are based on fully reversed behaviour. However, as stated on the Commentary of AS5100-6, the presented *S-N* curves are based on a conservative interpretation of data taken mostly from tests of structural elements under a wide range of conditions. The test results were interpreted in a graph with multiple *S-N* curves by means of different detail categories that implies the influence of the mean stress in fatigue behaviour of structures is reasonably covered.

A conservative definition of N_i is given in Eq.5.

$$N_i = 5 \times 10^6 \, (f_{3nc} / f_{in}^*)^5 \quad (5)$$

For shear stresses

$$N_i = 5 \times 10^8 \, (f_{5sc} / f_{is}^*)^5 \quad (6)$$

The fatigue design life on the structure in years L_y shall be taken as

$$L_y = 1/D \quad (7)$$

More details on how to determine the parameters used in this procedure are explained in AS5100.6 (2004) and BS 5400 Part 10: 1980.

It is noted that only passenger train loading was considered for the fatigue analysis as the number of heavy haul trains across the viaduct tracks has always been negligible. A conservative estimate revealed that 50% of 300LA load can be used to represent passenger train load. The remaining fatigue life was estimated based on the assumption that 200 passenger trains per day will pass over each track.

A result of this analysis will place a theoretical remaining life on the structure. This remaining life will not determine whether the viaduct will be structurally capable to withstand the loadings of the trains but rather is used as an indicator of time until fatigue defects start to develop**.**

The results shown in Table 4 revealed a theoretical remaining life of 24 years based on most the critical members. Therefore, without strengthening works, a monitoring program should be implemented after this time to assess the critical members for fatigue induced cracking.

Table 4. Fatigue life results for the superstructure at 200 trains per day on each track

Member (Critical)	Cumulative damage to date	Remaining fatigue life (Years)
Girder	0.7853	**24**
Rivets	0.0289	>100
Trough Deck	0.5349	78

As mentioned previously, during the inspection stage of this project, fatigue testing was carried out for a number of girders. Results of the fatigue testing indicated that there were no induced fatigue cracks to the tested area. The test results verified the desktop evaluation by confirming that the structure has not yet reached its fatigue life.

5 CONCLUSION

The following conclusion can be drawn from this investigation.

- Following the proposed methodology, old riveted bridges can be successfully evaluated for their As-Is condition under actual loading conditions.
- A detailed inspection and field tests are required to support analytical modeling.
- Bridges of this age can be still serviceable for an extended period of time if properly maintained.
- These bridges are most likely fail under new load requirements stipulated in modern design codes. However, applying more realistic loads based on the actual load condition can keep these structures in service with applicable load or speed restrictions.
- Fatigue assessment can reasonably predict the theoretical fatigue life of the structure. It is recommended to not rely solely on desktop assessment and seek verification by conducting field testing.
- The remaining fatigue life does not address suitability of the structure to withstand the loadings of the trains but rather is used as an indicator of time until fatigue defects start to develop**.** Once theoretical fatigue life is expired, the existence or development of fatigue defects should be monitored.

6 REFERENCES

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