Engineering Assessment of Steel Railway Bridges Based on Past Performance

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SYNOPSIS

The Australian Rail Track Corporation (ARTC) is responsible for over 2,200 railway bridges on its rail network which extends from Queensland border through New South Wales, Victoria and South Australia to Western Australia. Of these structures there are 780 transom top steel bridges, most of them built circa 1910.

The load carrying capacity of short span bridge elements such as RSJ girders, stringers and cross-girders do not comply with the current Bridge Design Code AS5100 at the ultimate limit state (ULS) in bending, yet these elements are performing without distress due to bending.

Performance assessment of a network representative sample of steel bridges was undertaken in accordance with AS13822 – “Basis for Design of Structures – Assessment of Existing Structures”, 2005. It was determined that all bridges are deemed safe under current traffic loading and speed.

This AS13822 compliance has structural design consequences on dynamic load allowance (DLA) and ULS code factors. The paper discusses in detail the impact of high code DLA for railway traffic loading comprising:

- Assessment in accordance with AS13822;
- A comparison of the DLA in AS5100 to the most relevant international standards;
- The interaction of DLA and Live Load factors;
- Factors contributing to railway DLA.

Keywords: Steel bridge, railway bridge, transom top bridge, open deck bridge, dynamic load allowance, assessment based on past performance.

INTRODUCTION

Majority of the existing steel bridges on ARTC network are approaching 100 years of age and were designed in accordance with British Standards. Between 2006 and 2008, 89 of them were load rated in accordance with AS5100.2 – Australian Standard – Bridge Design – Part 2: Design Loads, 2004 by engineering consultants, Pitt & Sherry Pty Ltd of Melbourne and Hughes Trueman Pty Ltd of Sydney, Australia.

The consultants concluded that short span open deck bridge elements such as RSJ girders, stringers and cross-girders do not have adequate load carrying capacity at full track speed due to high DLA factors in AS5100.2. Consequently this could lead to the imposition of train speeds on a large stock of structures on the ARTC network, significantly affecting operational efficiency.
A study of a network representative sample of bridges with these elements was conducted in accordance with AS13822. After inspection and assessment of 18 typical bridges, and rigorous application of AS13822 requirements, it was determined that all bridges are deemed safe under current traffic loading and speed. AS13822 deems a structure to be safe if current loading does not differ from historical loading, and if no changes are made to the structure.

This AS13822 compliance has structural design consequences on DLA and ULS code factors. A review of the DLA compared AS5100.2 to the most relevant following international standards and recent previous design standards in Australia:

- BS5400.2 – British Standard – Steel, concrete and composite bridges – Part 2: Specification for loads, 2006;
- NR/GN/CIV/025 – Network Rail – The Structural Assessment of Underbridges – Section 4: Loading for Assessment, June 2006 (Great Britain);
- Australian Bridge Design Code – Austroads – Section 2: Design Loads, 1992;
- ANZRC Railway Bridge Design Manual – Australia and New Zealand Railway Conferences – 1974 (Australia / New Zealand);
- Manual for Railway Engineering – American Rail Engineering and Maintenance – of - Way Association (AREMA), 2001 (United States of America);

The interaction of DLA and Live Load factors was also specifically assessed. Analysis has shown that in order to maintain the DLA values required under AS5100.2 for operational loading of Class 81/82 locomotives hauling 100 tonne wagons, the live load factor needs to be reduced to the range of 0.8 up to 1.4, depending on the section properties and span lengths of bridge members.

The paper discusses the effects of DLA for short span members by comparing the requirements of different national and international codes and the subsequent modification to ULS factors in order to maintain current operational speeds.

2 AS 13822

AS/ISO 13822:2005 assesses structures based on past performance with regards to safety and serviceability. The study addressed the following issues:

2.1 Assessment of Safety and Serviceability

AS/ISO 13822:2005 states that structures designed and constructed based on earlier codes, or designed and constructed in accordance with good construction practice when no codes applied, may be considered safe to resist actions other than accidental actions (incl. earthquakes) and serviceable for future use provided that:

- Careful inspection does not reveal any evidence of significant damage, distress or deterioration;
- The structural system is reviewed, including investigation of critical details and checking them for stress transfer;
• The structure has demonstrated satisfactory performance for a sufficiently long period of time for extreme actions due to use and environmental effects to have occurred;
• Predicted deterioration taking into account the present condition and planned maintenance to ensure sufficient durability;
• There have been no changes for a sufficiently long period of time that could significantly increase the actions on the structure or affect its durability, its use that could significantly alter the actions including environmental actions on the structure or part thereof and no such changes are anticipated.

3 ASSESSMENT BASED ON PAST PERFORMANCE

Arup Pty Ltd, consulting engineers of Sydney, assessed the existing structures based on past performance, in accordance with AS13822 criteria. All of the inspected bridges are deemed to comply.

To satisfy safety and serviceability requirements based on past performance under AS13822:2005, the five issues mentioned in Section 2.1 were addressed as follows:

3.1 Inspection

Thorough inspections revealed very little evidence of significant damage, distress, deterioration or section loss.

3.2 Review of Structural System

During the inspection process, the structural system was reviewed. This included the investigation of critical details, such as connections / bearings, and visual checking for normal stress transfer. Below are typical photos of the inspected bridges.

![Figure 1 – Typical simply supported RSJ short span bridge](image)
3.3 Past Performance

The historical rail traffic over the past twenty years has consisted of generally 81/82 class locomotives hauling 92 tonne wagons, which equates to roughly 920kN per wagon. It has also been determined that upper-limit axle loads for all bridges under this study are 23 tonne per axle, resulting in an axle loading of 230kN per axle.

Visual inspections have shown that none of the bridges have suffered any distress as a cause of this loading, nor as a result of any adverse environmental effects such as wind-loading and corrosion. Since all bridges have been in services for longer than twenty five years, the structures have demonstrated satisfactory performance for a sufficiently long time.

3.4 Predicted Deterioration

Taking into account the current state of the bridges inspected, and assuming regular routine maintenance, it is expected that the condition of the bridge elements will not degenerate in short to medium term. There are negligible sectional losses, and the connections are fully functional and in good condition.

3.5 Forecasted Structural Utility

As mentioned in section reference 3.3, 81/82 class locomotives and 92t wagons have consistently been in service for the past twenty five years, and it is not expected for these traffic loads to be exceeded in the near future. Another issue that could subject the structure to increased actions or endanger its durability is the modification of the structures. Again, no future changes to the structure, apart from routine maintenance or repair, are assumed.

4 COMPARISON OF DLA IN INTERNATIONAL CODES

The above AS13822 compliance has structural design consequences on DLA and ULS code factors. A review of the DLA comparing AS5100 to the most relevant national and international standards is as follows:

Dynamic loads are equivalent static loads that are multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by wheel and track irregularities. In AS5100.2, the DLA (or $\alpha$) is dependent on the characteristic length of a member ($L_\alpha$), as well as the method of track support, i.e. ballast or direct fixation (transom top). In the commentary to AS5100.2, AS5100.2 Supp 1: 2007, DLA is defined as “the dynamic load effect occurring with the member under consideration, with the type of wheel defect maximizing dynamic load effect for that member, with a wheel defect at the condemning limit. This wheel defect is defined as the limit beyond which a vehicle is not permitted to run.”

AS5100.2 Supp 1 also states that the relationship between DLA and train speed can be derived based on natural frequencies of the bridge, as well as the level of damping. These factors have however been simplified, and DLA factors are provided in an empirical form, as shown in Tables 8.4.3.1 (ballast) and 8.4.3.2 (direct fixation) in AS5100.2. Table 8.4.3.2 has been reproduced below.

<table>
<thead>
<tr>
<th>Characteristic Length ($L_\alpha$), m</th>
<th>Dynamic load allowance ($\alpha$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 2.0$</td>
<td>1.6</td>
</tr>
<tr>
<td>$&gt; 2.0$</td>
<td>$\frac{2.16}{L_\alpha^{0.5} - 0.2} - 0.17$</td>
</tr>
</tbody>
</table>

Table 1 - Values of for bending moment for open deck and spans with direct fixation

The maximum dynamic load is usually recorded with a train speed in the range of 80 to 100km/h or higher, with significantly worn disc-braked wheels. Since these relationships are empirical, they could be further refined taking into account vehicle speed and natural frequency of the structure (as is the case under Network Rail, ABDC and UIC codes) for a less conservative, but acceptable, DLA.

The design action caused by dynamic loading is equal to $(1+\alpha) \times$ the load factor $\times$ the action under consideration, e.g. when $\alpha = 1.6$ (max), then the design action caused by dynamic loading is equal to $2.6 \times$ the load factor $\times$ the action under consideration.

4.2 British Standard – BS5400.2 (2006)

Similar to AS5100.2, the DLA in BS5400.2 is based on length (L) of the “influence line of the element under consideration.” Values to be used for dynamic factors are found in Table 16 of BS5400.2, and are reproduced below. The dynamic load factors listed apply to all types of track.

<table>
<thead>
<tr>
<th>Dimension L, m</th>
<th>Dynamic factor for evaluating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bending Moment</td>
</tr>
<tr>
<td>Up to 3.6</td>
<td>2</td>
</tr>
<tr>
<td>From 3.6 to 67</td>
<td>$0.73 + \frac{2.16}{L^{0.5} - 0.2}$</td>
</tr>
<tr>
<td>Over 67m</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 2 - Dynamic factors for type RU loading (BS5400.2)
The allowances for dynamic effects have been calculated so that they cover slow moving heavy, and fast moving light vehicles. It is assumed that exceptional vehicles move at speeds not exceeding 80km/h, heavy wagons move at speeds up to 120km/h, and passenger trains move at speeds up to 200km/h.

For determination of the dynamic factor under a specific combination of speed and loading, the designer is referred to the UIC guide (leaflet 776-1R).

The design action caused by dynamic loading is equal to $\alpha \times$ the load factor $\times$ the action under consideration e.g. when $\alpha = 2.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.

### 4.3 Network Rail - NR/GN/CIV/025 (2006)

Network Rail’s guide, The Structural Assessment of Underbridges, provides a more detailed and flexible approach to the dynamic load allowance. For the purpose of this report, the “dynamic load increment ($\phi$)” nominated in the Network Rail guide, has the same definition as the “dynamic load allowance ($\alpha$)” nominated in AS5100.2 and BS5400.2.

Network Rail approaches dynamic loading from a different perspective to AS5100 and BS5400 in that it splits dynamic loading into two components, $\phi_1$ and $\phi_{11}$. These components are calculated separately, and combined to yield a single dynamic increment, $\phi$.

$\phi_1$ relates to the interaction of the structure and peaks when the speed of vehicles coincides with the natural frequency of vibration of the structure leading to a resonant condition. The formula for $\phi_1$ is theoretically based, conservatively assuming that structural damping is absent, with speed and natural frequency of the structure being the only parameters being considered.

$\phi_{11}$ covers the dynamic effects of track irregularities. The formula is based on results of tests leading to UIC code 776-1R (discussed in Section 4.7 of this paper). The formula has been adjusted so that when a short determinant length applied ($L_\phi$, similar to characteristic length mentioned previously) illogically high values of $\phi_{11}$ occur. Due to this reason, the minimum allowable value for $L_\phi$ in formulas is 4.0m. This can be seen from the design charts, mentioned below.

The Network Rail guide provides charts for values of $\phi$ for different structural frequencies, determinant lengths, and vehicle speeds. An example of one of these charts can be found in Figure 3.

The dynamic increments given in the design charts apply for directly-fixed tracks, and the guide makes allowance for reduced dynamic effects for ballasted decks.

It should be noted that the design action caused by dynamic loading is equal to $(1+ \phi) \times$ the load factor $\times$ the action under consideration. Therefore, as an example, when $\phi = 1.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.

The dynamic load allowance, $\alpha$, for railway live loading effects, is a proportion of the static railway live load. It has the same values for structures of reinforced/prestressed concrete, steel and composite construction. Unlike the Network Rail guide discussed earlier, $\alpha$ is the same for both ballasted and direct-fixed tracks. In this code, the value of $\alpha$ is based solely on the characteristic length, $L_{\alpha}$.

The following table gives estimated values of $\alpha$ for bending moment calculations:

<table>
<thead>
<tr>
<th>Characteristic Length ($L_{\alpha}$), m</th>
<th>Dynamic load allowance ($\alpha$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\leq 3.6$</td>
<td>1.0</td>
</tr>
<tr>
<td>$3.6 &lt; L_{\alpha} &lt; 67$</td>
<td>$\frac{2.16}{L_{\alpha}^{0.5}} - 0.27$</td>
</tr>
<tr>
<td>$&gt; 67$</td>
<td>0</td>
</tr>
</tbody>
</table>

*Table 3 - Values of DLA for bending moment, $\alpha$ (Table 2.4.6.3 – ABDC 1992)*

Clause 2.4.6.3 states that “The value of $\alpha$ for bending moment shall be determined according to Table 2.4.6.3” (shown above). This means that although the value of $\alpha$ is an estimate / simplification of more detailed calculations outlined in the commentary to this code, the values from the table must be used.
However, Clause 2.4.6.4 states that “where detailed information is available for specific structures and track standard, and where train speeds are known, $\alpha$ may be calculated for the procedures described in the commentary”. The approach found in the commentary is the same approach as the Network Rail Guide, in that it splits DLA into two components; the dynamic increment of a geometrically perfect structure, and the dynamic increment due to track irregularities. Formulation under these two standards is identical taking into account imperial-metric conversions.

Note that for our general design axle loading of 25t, the train design speed for the tabled DLA is 120km/hr. The design action caused by dynamic loading is equal to $(1+\alpha) \times$ the load factor $\times$ the action under consideration, e.g. $\alpha = 1.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.

### 4.5 ANZRC Railway Bridge Design Manual (1974)

The dynamic load allowance in the ANZRC Manual is defined as an “impact load”. The impact load is determined by taking a percentage of the live load, and is applied vertically and equally at the top of each rail.

The following formulae are presented for determining impact load percentages on open deck direct fixed rail tracks, with one track loaded, and for steel structures only.

For ballasted-deck bridges, this factor is multiplied by 0.9.

- \[ \text{For spans less than 25m, Impact percentage} = \frac{31}{Y} + 40 - \frac{3L^2}{150} \]
- \[ \text{For spans greater than 25m, Impact percentage} = \frac{31}{Y} + 16 + \frac{183}{L - 10} \]

Where $L$ is the span length, in meters, centre to centre of supports for stringers and $Y$ is the distance, in meters, between centres of single or groups of longitudinal beams, girders or trusses; or length between supports of floor beams or transverse girders.

All subject bridges are single track, but in the case of loading on more than one track, the following criteria apply:

- For $L$ less than 55m: Full impact on two tracks
- For $L$ between 55m and 70m: Full impact on one track and a percentage of full impact on the other as given by the formula $450 - 6.5L$
- For $L$ greater than 70m: Full impact on one track, and none of the other
- For more than two tracks loaded, and for all values of $L$: Full impact on any two tracks.

The design action caused by dynamic loading is equal to $(1+\alpha) \times$ the load factor $\times$ the action under consideration, e.g. when $\alpha = 1.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.
The AREMA Bridge Code provides dynamic load allowance based on trains with, or without, engine hammer blow.

Differentiation of DLA for both of these situations is relatively constant, with DLA only decreasing slightly with an increase in span length. Maximum DLA ($\alpha$) for trains with hammer blow is 0.8, and the maximum for trains without hammer blow is 0.6. The differentiation in DLA is similar to that in the ANZRC code (Section 4.5).

![Figure 4 - Excerpt from AREMA Manual 15.1 (RE = Rocking Effect)](image)

It should be noted that the design action caused by dynamic loading is equal to $(1+\alpha) \times$ the load factor $\times$ the action under consideration. Therefore, as an example, when $\alpha = 1.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.

4.6 UIC – International Railway Bridge Design Code

The UIC code (Leaflet 776 -1 R) is the code that is directly referenced by the Network Rail Guide (Section 4.3), and the ABDC (Section 4.4), and serves as the basis for many other codes. The UIC code provides a table of non-specific baseline arbitrary values for dynamic load factors for different span lengths and track conditions for the design of new bridges.

The values are based solely on characteristic length, and no distinction is made between the various methods of carrying track (i.e. with or without ballast). The baseline values are as follows.
When it comes to the assessment of specific existing bridges, the UIC code allows the use of its commentary, which is similar to the Network Rail Guide, and the ABDC. The UIC commentary permits the engineer to determine the dynamic load allowance based on the natural frequency of the structure, vehicle speed, and condition of tracks.

The dynamic load factor is split into two separate entities:
- the dynamic increment of a geometrically perfect structure;
- the dynamic increment due to track irregularities.

Calculation of the two separate increments is a similar methodology to Network Rail Guide and ABDC provisions. Results of a sample calculation of the two separate increments are provided in Table 4 (value of 0.91 and 0.72 respectively). The following figure is the guidance under UIC for determination for natural frequency for this specific assessment. It should be noted that the limiting span is 4 metres.

The design action caused by dynamic loading is equal to $\varphi \times$ the load factor $\times$ the action under consideration, e.g. when $\varphi = 2.0$, then the design action caused by dynamic loading is equal to $2.0 \times$ the load factor $\times$ the action under consideration.
5 Discussion

5.1 Short Span Twin Girder Bridges

5.1.1 General Description

A short span twin girder bridge, as shown in Figure 1, is a bridge where the rail and bridge superstructure are supported by two girders without intermediate cross-girders. These girders are considered to be simply supported, spanning between two supports. The spans of the inspected twin girder bridges are typically less than 4m.

5.1.2 Differences in International Codes

Consider a typical 3.1m short span girder bridge, being subject to railway traffic at speed of 100km/h. Assume that the natural frequencies of the bridge are 40.6Hz (high frequency, conservative upper limit) and 25.8Hz (low frequency, lower limit), and are extrapolated from the graph (see Figure 6), which represents the UIC formulation. Assume that the track is not maintained to exacting standards, is directly fixed to the bridge and there is no hammer blow effect from trains.

Table 4 shows the differences in DLA using approaches from the various codes and by percentage they are lower than AS5100.2 DLA. It can be seen that AS5100.2 specifies the most structurally punishing DLA.
AS5100.2 and BS5400 provide DLA factors based on inherently conservative assumptions founded on testing and literature. These assumptions include vehicle speed, vehicle load, and the condition of track and rolling stock. However, there is no latitude to modify these figures based on test data and other information.

In contrast to the above, the Network Rail, ABDC and UIC codes allow the designer to modify the dynamic load allowance based on the natural frequency of the structure, vehicle speed, and condition of tracks. As can be seen in Table 4, using an upper limit frequency of 40.6Hz and vehicular speed of 100km/h, the DLA can be calculated to be 0.72, almost 27% less than in AS5100.2. Using the lower bound frequency, the DLA is calculated to be as low as 0.91; 42% less than AS5100.2.

The ANZRC manual allows an even lower DLA (0.56), which is roughly 55% less than the recommended DLA value provided in AS5100.2. The AREMA code provides similar values to the ANZRC manual.

Where typical values for DLA are given and must be adhered to, such as in the case of AS5100.2 and BS5400, these DLA values are based on a particular loading, at a particular speed. These are documented on a worst case scenario, encompassing the worst case speed, loading and track condition. Therefore, it is reasonable to say that if the vehicle speed, loading and track conditions differ from the base assumptions of the code, that lesser values calculated under other codes may be tailored to specific conditions (including natural frequency).

### 5.1.3 Calculations

The effects of DLA on short span (< 4m) bridges when assessed against AS5100.2 reduced ULS live load factor to 1.4 from 1.6 so that the structure can adhere to AS5100 DLA requirements for maximum vehicle speed.

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1 This value is a calculated value based on train speed, natural frequency and track condition. If this information is not available, a value of 1 shall be adopted.

<table>
<thead>
<tr>
<th>Standard</th>
<th>DLA (α) at 40.6Hz</th>
<th>% &lt; AS5100.2 DLA at 40.6Hz</th>
<th>DLA (α) at 25.8Hz</th>
<th>% &lt; AS5100.2 DLA at 25.8Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS5100.2 - 2004</td>
<td>1.24</td>
<td>-</td>
<td>1.24</td>
<td>-</td>
</tr>
<tr>
<td>BS5400 - 2006</td>
<td>1</td>
<td>19.4%</td>
<td>1</td>
<td>19.4%</td>
</tr>
<tr>
<td>Network Rail - 2006</td>
<td>0.91</td>
<td>26.6%</td>
<td>0.72</td>
<td>42%</td>
</tr>
<tr>
<td>ABDC - 1992</td>
<td>0.91⁵</td>
<td>26.6%</td>
<td>0.72</td>
<td>42%</td>
</tr>
<tr>
<td>AREMA - 2001</td>
<td>0.6</td>
<td>51.6%</td>
<td>0.6</td>
<td>51.6%</td>
</tr>
<tr>
<td>ANZRC - 1974</td>
<td>0.56</td>
<td>54.8%</td>
<td>0.56</td>
<td>54.8%</td>
</tr>
<tr>
<td>UIC - 1994</td>
<td>0.91</td>
<td>26.6%</td>
<td>0.72</td>
<td>42%</td>
</tr>
</tbody>
</table>

Table 4 - Comparison of DLA from various codes
5.2 Short Span Stringer Members in Long Span Bridges

5.2.1 General Description

In contrast to a twin girder bridge, a through-girder involves the interaction of bridge elements to distribute and bear load. As can be seen in Figure 2, stringer beams lie directly underneath the rail, and loads from the stringers are transferred to the main girders by means of cross girders. In through girder and truss bridges, cross girders and stringers are typically short. In addition to this, stringers do not span in a simply supported manner, and are fixed at both extremities by bolts, welds and/or rivets.

Several end stringer members as elements of large span through girders that were encountered during inspections of the sample bridges were found to range from 2.1m to 2.75m. In accordance with Table 8.4.2 in AS5100.2; the characteristic length of end stringer members is that of the cross girder spacing.

5.3 End Stringer Members of Truss Bridges

The rating factors of end stringer members shorter than 6m are slightly lower than ULS live load factor of 1.6 thus necessitating train speed restrictions.

6 FACTORS CONTRIBUTING TO DLA

6.1 Natural Frequency

It is important to note that the natural frequency of the bridge structure has a large effect on determining the dynamic load allowance using the non-empirical methods set out in the Network Rail Guide, UIC, and the ABDC. When considering the example provided in Section 5.1.2, if the empirical values set out in UIC were to be used (similar to AS5100 approach), this would yield a dynamic factor of 2.0. When using UIC formulation however; a dynamic factor in the range of 1.72 and 1.91 (depending on the natural frequency of the structure) results.

6.2 Track Condition

Many codes allow the consideration of track condition while calculating the dynamic load factor. When considering the example provided in Section 5.1.2 and assuming exacting standards for track condition, the ABDC allows a 50% reduction in dynamic increment due to track irregularities. This yields a DLA of 0.46 (cf 0.91 or 0.72).

6.3 Damping

The effect of damping is ignored by the different bridge codes. This disregards the damping effects that timber transoms and elastomeric bearings have on short span bridges, and the effect of supporting bridge elements on modular structures with short span members.
7 CONCLUSION

Based on the requirements of AS13822, as discussed in Section 2 of the paper, the assessed bridges are deemed to satisfy all requirements. This signifies that the bridges may be considered safe to resist actions other than accidental actions (including earthquakes).

The actions being resisted are current vehicles in operation, travelling at current operable speeds, consistently in line with historical data for the past 25 years. As long as operating traffic does not differ from historical loading, and the bridge structure is not modified, the subject bridges are be fit for future use according to AS13822.

Although the bridges meet AS13822 requirements, there are certain structural adequacy implications regarding DLA and Live Load factors. Under the UIC code, for a bridge span of 3.48m with an operating speed of 100km/h, the applicable DLA ranges from 0.72 and 0.91, depending on natural frequency of the bridge. This results in a live load factor of between 1.7 and 1.6 respectively, whereas under AS5100, the DLA is 1.13, which results in a live load factor of 1.4.

8 SUMMARY

- All sample bridges assessed under the study comply with AS13822;
- Conclusion can be drawn that all bridges with similar short spanning members to sample bridges comply with AS13822 provided their original physical and structural integrity have not been altered significantly and similar traffic conditions prevail;
- The bridges will require reassessment if the subject members are modified, or if traffic loading increases significantly from the historical load;
- DLA under AS5100 has been proven to be the most conservative when compared to international standards;
- Should DLA under AS5100 be maintained the ULS factors need to be reduced in order to align with international standards as discussed earlier in the paper;
- Alternately, a more realistic set of DLA and/or load factors should be considered for load rating of the existing steel structures in accordance with AS5100.2.

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