AS(AS/NZS) 5100 - 2017: Rail Bridge Design and Rating - Key Changes

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Abstract: The revision of AS(AS/NZS) 5100 series has resulted in key changes to the provisions for train braking loads, dynamic load allowance, train collision loads, bridge bearings and design for light rail bridges. Longitudinal rail force provisions have been modified with revisions to the existing empirical formulae and the introduction of an alternative rational method for deriving these forces for complex bridge arrangements, and rail loading patterns. The rational method uses a first principles approach for determining the longitudinal rail forces applied to a bridge which can result in significant economies for long bridges in particular. The dynamic load allowance (DLA) has been revised by considering international standards and a recent study by ARTC. The maximum DLA value has been reduced from 160 down to 67% for bending stress. The recent pot bearing failures necessitated more stringent durability requirements for rail bridge bearings. New provisions have been introduced for light rail bridges. This paper discusses the new provisions and how to implement them.

Key words: Safety in Design, Rail bridge, railway loading, light rail, dynamic load allowance, longitudinal force, traction force, collision load, spherical bearing, SFAIRP.

Introduction

A suite of revised Australian Standards/New Zealand Standards Bridge Design Code AS(AS/NZS) 5100 was published in March 2017.

The requirements for design of rail bridges are now fully embedded in this series with the primary objective of ensuring reliable and safe bridges for the rail passengers and operators.

The revision follows an extensive review of all Parts of the series and addition of new ‘Part 8: Rehabilitation and strengthening of existing bridges’ and ‘Part 9: Timber bridges’. The revised Code enables designers of new bridges and maintenance of the existing bridges to better comprehend from well-defined Code to support them with structures designs.

Some significant changes have been made to all the parts of the Code but this paper only addresses significant changes made to rail traffic loading in Part 1: Bridge Design, Part 2: Design Loads, Part 4: Bearings and deck joints and Part 7: Bridge Assessment.

The primary changes made are as follows:
1. Inclusion of “Safety in Design” requirements
2. Inclusion of “Light Rail”
3. Clarification on “Surcharge Loads from Rail Traffic”
4. Modifications to “Dynamic Load Allowance for Rail bridges”
5. Additions to “Bridge Assessment” (Load Rating)
6. Modifications to “Braking and Traction Forces”
7. Modifications to “Train Collision Loads”
8. Durability enhancement of “Rail Bridge Bearing”.

1. Safety in Design

Safety in Design is mandated by statutory requirements and is required to be incorporated into the design. Here’s excerpt from Part 1:
“The safety in design process shall identify potential hazards and the potential risks to people during construction, future operation, maintenance and eventual decommissioning of an asset. During the design phase, risks shall be eliminated or minimized. The risks shall be ranked before and after a suitable control has been selected for implementation, which shall highlight where sufficient risk reduction controls have been achieved through design to reduce the risk to an acceptable level, using a process approved by the relevant authority. Residual risks shall be managed within the asset's life cycle.

The designer shall document, as a minimum, the following:

a) Hazards identified associated with a design (e.g. hazardous structural features, hazardous construction materials and hazardous procedures or practices) that might be realized in the construction, operation, maintenance and decommissioning phases of the project life cycle.

b) The hazard in terms of the potential risks of injury or harm.

c) The mitigation controls the designer has developed or utilized to reduce any risk.

d) Aspects of the design where the hazard has been identified but cannot be resolved during the design and needs to be managed during the construction, operation, maintenance and/or decommissioning phases”.

There are a few commendable guidelines on safety in design, notably the following:

- “Safe Decisions – a framework for considering safety when making decisions in the Australian Rail Industry” guideline by Rail Industry Safety and Standards Board (RISSB), March 2016.
- “Meaning of Duty to Ensure Safety So Far As Is Reasonably Practicable” guideline by Office of the National Rail Safety Regulator, January 2014.

2. Light Rail

The Code (2017) includes design traffic load for Light Rail. The Light Rail design load is based on 150LA. It is derived by multiplying the 300LA design load by 0.5 and, is limited to the first 9 axles only. The ‘LA’ configuration was chosen so that the other associated horizontal forces such as centrifugal, braking, traction and nosing forces could be derived by use of the provisions within AS 5100 and proportioning the forces by the ratio of the axle loads. The distribution of Light Rail axle loading is the same as in AS 5100 for 300LA rail traffic.

The design load was derived by comparing current Light Rail vehicles operating in Melbourne and Sydney. The most common vehicles comprise 4, 6 or 8 axles with lengths varying from about 20 to 33m for the 4 and 8 axle sets respectively, thus restricting the total axles to 9 only. The axle loads vary from about 11 to 13.5 tonnes.

A 100 year growth factor of 1.3 (30% increase) was added to the current Light Rail vehicles. In addition, tandem vehicles were allowed for. Figure 1 below depicts a graph comparing the factored bending moments for the current light vehicles for simply supported span lengths from 1 to 60m. Similar graph results for shear. The bending moments and shear forces are compared to proposed design load of 9 axles from 150LA.

![Figure 1: Comparison of Light Rail Vehicle Bending Moments](image)
3. Surcharge Loads from Rail Traffic

The current provisions in AS 5100 with respect to surcharge from rail traffic have not changed. Tabulated values have been included in AS 5100 Table 14.3 to assist in the determination of the variation of vertical surcharge pressure (kPa) with depth for design traffic.

In AS 5100, a line of influence of slope 1 to 1 is adopted starting from the underside of the edge of the sleeper. If the retaining wall or abutment lies within this line of influence then surcharge is applied. In calculating the surcharge at the underside of the sleeper, the area of surcharge is defined by the width of sleeper and the length of loaded track. The increasing area with depth at a rate of 1H to 2V in both the transverse and longitudinal direction is assumed. In the transverse direction, the distribution width shall not exceed 4.5m. In the longitudinal direction, the loaded track shall extend 0.55m beyond the end axle. Figure 2 depicts the distribution in the transverse direction. Figure 3 depicts an example for the combination of 300LA axles 1 to 5 for the distribution length in the longitudinal direction. Other axle combinations (e.g. A2+A3, A2+A3+A4, etc.) have been considered in the table to produce the highest pressure.

Figure 2: Lateral pressure distribution beneath a sleeper

Figure 3: Longitudinal pressure distribution beneath a sleeper
4. Dynamic Load Allowance for Rail Traffic ($\alpha$)

In Part 2 of AS 5100 (2017), the dynamic load allowance (DLA) has been revised by considering international standards and a study undertaken by Australian Rail Track Corporation (ARTC) in 2008.

The maximum DLA value has been reduced from 160 down to 67% for bending moment. The load carrying capacity of short span bridge elements such as RSJ girders, stringers and cross-girders were significantly affected by high DLA to comply with the previous Bridge Design Code AS 5100 (2004) at the ultimate limit state (ULS) in bending, yet these elements were performing without distress due to bending.

A short span twin girder bridge, as shown in Photo 1 below, is a bridge where the rail track is supported by two “I” girders without intermediate cross-girders. These girders are considered to be simply supported, spanning typically less than 4m between two supports.

**Photo 1:** Typical simply supported RSJ short span bridge

**Photo 2:** Typical through span with short span stringers and cross girders

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 Photo 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 span bridges, cross girders and stringers are typically short, ranging from 2 to 4m. 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.

Majority of the existing steel bridges on ARTC network are approaching 100 years of age and were designed in accordance with British/American Standards. Between 2006 and 2008, 89 of them were load rated in accordance with AS 5100.2 – 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 above 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 AS 5100.2 (2004). Consequently this could lead to the imposition of train speeds on a large stock of structures, significantly affecting operational efficiency.

A study of a network representative sample of bridges with these elements was conducted by ARUP Pty Ltd [11] for ARTC in accordance with AS 13822 [9]. After inspection and assessment of 18 typical bridges, and rigorous application of AS 13822 requirements, it was determined that all these bridges were deemed safe under current traffic loading and speed. AS 13822 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 AS 13822 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:


**Australian Standard - AS5100.2 (2004)**

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 is dependent on the characteristic length of a member (Lα), as well as the method of track support, i.e. ballast or direct fixation (transom top). In the commentary to AS 5100.2, AS 5100.2 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.”

AS 5100.2 Supplement 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 had 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 (Lα), m</th>
<th>Dynamic load allowance (α)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 2.0</td>
<td>1.6</td>
</tr>
</tbody>
</table>
| > 2.0                        | \[
\frac{2.16}{L_{α}^{0.5}} - 0.17
\] |

**Table 1:** Values of for bending moment for open deck and spans with direct fixation (Table 8.4.3.2 - AS5100.2)

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

**British Standard – BS5400.2 (2006)**

Similar to AS 5100.2, the DLA in BS 5400.2 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 factors listed apply to all types of track.
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).

**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 DLA. For the purpose of this report, the “dynamic load increment (φ)” nominated in the NR’s guide, has the same definition as the “dynamic load allowance (α)” nominated in AS 5100.2 and BS 5400.2.

Network rail splits dynamic loading into two components, φ₁ and φ₁₁. These components are calculated separately, and combined to yield a single dynamic increment, φ.

φ₁ 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 φ₁ is theoretically based, conservatively assuming that structural damping is absent, with speed and natural frequency being the only parameters being considered.

φ₁₁ covers the dynamic effects of track irregularities. The formula is based on results of tests leading to UIC code 776-1R. The formula has been adjusted so that when a short determinant length applied (L₀, similar to Lₐ mentioned previously) illogically high values of φ₁₁ occur. Due to this reason, the minimum allowable value for L₀ in formulas is 4.0m.

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.

**Austroads – Australian Bridge Design Code (1992) – Railway Supplement**

The dynamic load allowance, α, 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, α is the same for both ballasted and direct-fixed tracks. In this code, the value of α is based solely on the characteristic length, L₀.

<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 + 2.16/Lₐ^{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 dynamic load allowance, α, 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, α is the same for both ballasted and direct-fixed tracks. In this code, the value of α is based solely on the characteristic length, L₀.

<table>
<thead>
<tr>
<th>Characteristic Length (L₀), m</th>
<th>Dynamic load allowance (α)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤ 3.6</td>
<td>1.0</td>
</tr>
<tr>
<td>3.6 &lt; L₀ &lt; 67</td>
<td>2 .16 / L₀^{0.5} - 0 .20 - 0 .27</td>
</tr>
<tr>
<td>&gt; 67</td>
<td>0</td>
</tr>
</tbody>
</table>

**Table 3**: Values of DLA for bending moment, α (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.

**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.

- For spans less than 25m, Impact percentage = $\frac{31}{Y} + 40 - \frac{3L^2}{150}$
- 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. In the case of loading on more than one track, the impact factor is adjusted on adjoining tracks.

**AREMA – Manual for Railway Engineering**

The AREMA Bridge Code provides DLA 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 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. Below is excerpt from AREMA Manual 15.1 (RE = Rocking Effect).

(1) Percentage of live load for rolling stock without hammer blow (diesels, electric locomotives, tenders alone, etc.):

a) For $L$ less than 80 feet: $RE + 40 - \frac{3L^2}{1600}$

b) For $L$ 80 feet or more: $RE + 16 - \frac{600}{L-30}$
**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 and the ABDC 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).

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 and 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.

![Natural frequency limits for unloaded bridges](image)

**Figure 4: Natural Frequency Limits for Unloaded Bridges (UIC 776-1R)**

**Differences in International Codes**

Consider a typical 3.1m short span girder bridge, being subject to rail 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 4), 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 AS 5100.2 DLA. It can be seen that AS 5100.2 specifies the most structurally punishing DLA.
<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>0.91(^1)</td>
<td>26.6%</td>
<td>0.72(^1)</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

AS 5100.2 and BS 5400 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 AS 5100.2. Using the lower bound frequency, the DLA is calculated to be as low as 0.91; 42% less than AS 5100.2.

The ANZRC manual allows an even lower DLA (0.56), which is roughly 55% less than the recommended DLA value provided in AS 5100.2. The AREMA code provides similar values.

Where typical values for DLA are given and must be adhered to, such as in the case of AS 5100.2 and BS 5400, 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).

**FACTORS CONTRIBUTING TO DLA**

The primary factors contributing to DLA are as follows:

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, if the empirical values set out in UIC were to be used (similar to AS 5100 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.

2. **Track Condition**

   Many codes allow the consideration of track condition while calculating the dynamic load factor. 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).

\(^1\) 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.
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.

**Revised DLA**

The revised DLA formula is same as in the previous AS 5100.2 for bending moment for ballasted deck spans but with upper and lower limits rather than restricting by \( L_a \).

\[
\alpha = \left\{ \begin{array}{ll}
2.16 & \text{for} \ L_a \leq 0.20 \\
0.27 & \text{for} \ L_a > 0.20
\end{array} \right. \leq 0.67
\]

*Figure 5: Excerpt from AS/NZS 5100 (2017)*

Extensive investigations were undertaken by AREMA to establish rail traffic impact factor. AREMA also concluded that impact is induced by rocking effects created by the transfer of load from the wheels on one side of a car or locomotive to the other side from periodic lateral rocking of the equipment. They limited the total impact factor with 0.20 rocking effects factor to 0.60 which the rail industry in Australia used until the introduction of Austroads – Australian Bridge Design Code (1992) – Railway Supplement, 1996 [3]. AS 5100 (2004) further intensified the DLA limit from 1.0 to 1.60. From Table 4 above, it can be seen that AS 5100.2 specifies the most structurally punishing dynamic load allowance.

Today, the rail tracks are maintained to exacting standards by regular monitoring and utilisation of modern track machines. Also, wheel impact load detectors have been installed along major freight routes to pick out defective wheels. The train operators also ensure their rolling stocks are well maintained to provide reliable services to their customers.

The upper limit of 0.67 in revised DLA was derived by averaging DLA values at 25.8Hz obtained using approaches from Network Rail, ABDC, AREMA, ANZRC and UIC in Table 4 above. The lower DLA value of 0.20 is based on influence from rocking effects of rolling stocks. In between the limits, the previous formula for bending moment for ballasted deck spans was retained for all deck types.

The revised DLA was derived by considering international standards, a study undertaken by ARTC and primary factors contributing to DLA. The BD-090 committee determined in light of the above evidence that it was reasonable to adopt the above revised DLA.

5. **Load Rating**

‘Part 7: Bridge Assessment’ was formally known as ‘Rating for existing bridges’. The significant changes made here apart from the title are as follows:
(i) **Methodology:** Overall there is a clearer methodology for bridge assessment and load rating including a step-by-step guide and flowchart.

(ii) **Load rating vehicles:** Improved definition and clarity have been provided regarding vehicles used for bridge load rating.

(iii) **Structural capacity:** Greater detail and guidance have been provided including data collection, material properties and considerations and assessment methods.

(iv) **Structural health monitoring:** This has been included in the Standard to provide an overview of the latest technology and guidance on its potential use for bridge assessment.

(v) **Historical material properties:** Information on past material Standards and properties has been provided to assist bridge assessors in understanding the probable material properties of bridges built to previous Standards.

6. Braking and Traction Forces

The provisions for defining and distributing rail longitudinal forces resulting from braking and traction forces in AS 5100–2004 have been revised in the 2017 code, principally for the following reasons:

1) To reflect the results of research and investigations into these forces that have been conducted that had not been incorporated into the AS 5100–2004 provisions; and

2) To account effectively for the scope of rail vehicle loadings and bridge configurations that typically occurs in the Australian rail networks.

In the 2017 code, the applied braking and traction forces can be derived by either of two methods, namely the empirical method or the rational method.

**Empirical Method**

In the empirical method, the forces are calculated using revised formulae, noting that:

- no differentiation is made between continuously welded rail and discontinuous rail bridges;
- braking forces are based on 15% wheel-track adhesion and traction forces on 50% adhesion;
- the forces so derived are the nett forces imposed on the bridge having already taken account of longitudinal forces transferred off the bridge; and
- each occupied track is to be loaded for its full length with 300LA rail traffic loading.

**Rational Method**

The rational method was introduced into the code as an effective method of dealing with the range of rail traffic and the ever increasing sophistication of rail bridge solutions appearing on Australian rail networks. In this method, the loadings applied to bridges are derived from a set of bridge specific design parameters stipulated by the rail authorities responsible for the bridges. An example of a Rail Authority’s “Bridge Specific Design Parameters” for distribution of forces based on Rational Method is presented below.

<table>
<thead>
<tr>
<th>RAIL VEHICLE</th>
<th>Rail Traffic Loading (LA)</th>
<th>CoG above top of rail (m)</th>
<th>Length (m)</th>
<th>Traction Acceleration (m/s²)</th>
<th>Traction Length (m)</th>
<th>Braking Deceleration (m/s²)</th>
<th>Braking Length (m)</th>
<th>Min. Clear Distance between Trains (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FREIGHT TRAIN</td>
<td>300</td>
<td>2.1</td>
<td>1200</td>
<td>0.5</td>
<td>120</td>
<td>1.5</td>
<td>1200</td>
<td>1200</td>
</tr>
<tr>
<td>PASSENGER</td>
<td>245</td>
<td>1.9</td>
<td>163</td>
<td>0.5</td>
<td>163</td>
<td>0.75</td>
<td>163</td>
<td>300</td>
</tr>
<tr>
<td>WORK TRAIN</td>
<td>245</td>
<td>2.1</td>
<td>300</td>
<td>0.5</td>
<td>25</td>
<td>0.75</td>
<td>300</td>
<td>300</td>
</tr>
</tbody>
</table>

**Table 5:** Example of a Rail Authority’s “Bridge Specific Design Parameters”

The longitudinal forces derived from the above methods are to be distributed through bridge structures using valid rail-structure interaction models approved by the relevant rail authority. It is important to note that for bridges analysed using empirical method loadings, the track form shall be modelled for the extent of the
bridge deck only, whilst for bridges analysed using rational method loadings the track form and the appropriate rail vehicle loadings shall be modelled for 100 metres minimum beyond each bridge abutment.

7. Train Collision Loads

The provisions for derailed train collision have been revised and clarified. The intent of the provisions is to prevent high loss of life and bridge collapse resulting from train derailments for both rail bridges and overpasses. The Committee considered world’s best practice and the possible precautions that can be taken to avoid major disasters. The new provisions are considered to be the world’s most advanced and challenging safety in design guidelines with the overriding focus on satisfying the intent of the provisions even for the more complex bridge designs. The provisions are discussed in detail in Rapattoni 2017 paper [12].

8. Rail Bridge Bearing

Recently documented reports by ARTC of premature-failures of pot bearings due to elastomer extrusion have prompted detailed investigations into the root causes of the problem. The results of the investigations, particularly related to the durability of various internal-seal systems used in the bearing-industry, lead to changes to AS/NZS 5100.4. The changes are to assist bridge designers and managers in selecting bearings with reliable seal systems, enabling significant reduction in total life-cycle costs. The results of investigation and need for more stringent durability requirements for rail bridge bearings are presented in Prasad 2017 paper [13].

9. Conclusion

The revised bridge Code AS(AS/NZS) 5100: 2017 enables bridge designers to better comprehend from well-defined Code to support them with structures designs. The requirements for the design of rail bridges are now amply embedded in the series with primary objective of ensuring reliable and safe bridges for the rail passengers and operators.

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References