UGL REGIONAL LINX

BRIDGES - LOAD RATINGS

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CRN CM 308

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Table of Contents

Document Control

Function	Position	Name	Date
Approver	A&E Manager	Lucio Favotto	24.01.2022

Revision	Issue Date	Revision Description
1.1	25.11.2021	UGLRL Operational Standards Template applied
2.0	09.12.2021	First approved and issued UGLRL version
3.0	24.01.2022	Issued for publish to intranet and webpage.

Summary of changes made from previous version

Section	Summary of change
All	This document is based on the previous rail infrastructure maintainer (RIM). Full revision history is available on request from UGLRL

Chapter 1 Introduction

C1-1 Purpose

The purpose of the manual is to set out requirements and procedures for the load rating of bridges on the Country Regional Network (CRN).

The manual describes a framework to determine a qualitative load rating, based on past performance, and provided intervention levels that will determine the need for an engineering load rating, and or, further assessment.

The manual is developed for the assessment of all operational bridges to ensure capability to withstand current road, rail and pedestrian loadings. The manual covers a variety of bridge construction materials including:

- Reinforced Concrete
- Prestressed Concrete
- Steel
- Timber
- Wrought Iron
- Cast Iron
- Masonry

The methodology included in this manual draws on that which has been used and refined in the rating of bridges on the NSW rail network in recent years. It therefore formalises previous procedures and ensures compatibility of future work.

Assessment of the remaining life of the bridge, related to fatigue effects, is not included.

C1-2 Context

The manual is part of UGLRL CRN's engineering standards and procedures publications.

More specifically, it is part of the Civil Engineering suite that comprises standards, installation and maintenance manuals and specifications. Manuals contain requirements, processes and guidelines for the management of track, structures, geotechnical and right of way assets and for carrying out examinations, construction, installation and maintenance activities

The manual is written for the persons assessing existing bridges and is based on the principles of structural reliability and consequence.

C1-3 Who should use this manual

This manual should only be used by professional engineers, experienced in bridge design and assessment who have been granted the appropriate Engineering Authority by the Principal Track and Civil Engineer. The procedure is applicable to the assessment of bridges that were originally designed, analysed and specified based on design standards, current at the time of construction.

The assessment can be initiated under the following circumstances:

- Reliability checks for on-going use
- Prior to an anticipated change in use
- Following structural deterioration due to time-dependent actions (e.g. fatigue, loss of section due to corrosion)
- Following structural damage by accidental actions

C1-4 Background

The Australian Bridge Design Code AS 5100 includes the design and rating of bridges. This includes both road bridges over railways (overbridges) and bridges carrying railway loadings (underbridges). In particular, Section 7 of the Code covers the rating of existing bridges. This approach to bridge rating adopts a Limit States format in contrast to previous documents in use, viz, ANZRC Railway Bridge Design Manual (1974), AREA Railway Engineering Manual (1984) and the NAASRA Bridge Design Specification (1976).

This manual is compatible with AS 5100. It also closely relates to procedures used by former rail organisations (Freight Rail, RSA and RIC) to load rate vast numbers of bridges on the CRN. Rating of bridges is to be in accordance with the AS 5100 Section 7 and other applicable AS standards.

C1-5 References

C1-5.1 Australian and international standards

AS/NZS 1170 - Structural design actions

AS 1391 - Metallic materials - Tensile testing at ambient temperature

AS 1720 - Timber Structures

AS ISO 13822 - Basis for design of structures – Assessment of existing structures

AS 3700 - Masonry structures

AS 4100 - Steel structures

AS 5100 - Bridge design

C1-5.2 CRN documents

CRN CS 100 – Civil Technical Maintenance Plan

CRN CS 300 - Structures System

CRN CS 310 – Underbridges

CRN CS 320 - Overbridges

CRN CM 001 – Civil Technical Competencies and Engineering Authority

CRN CM 302 – Structures Examination

CRN CM 305 – Structures Assessment

CRN CM 307 – Bridge Assessment Procedure for Heavy Vehicles

Unless otherwise specified, all references relate to the latest standard versions, including amendments and relevant superseding standards.

C1-5.3 Other references

Australian and New Zealand Railway Conferences (ANZRC) Railway Bridge Design Manual 1974

RMS Bridge Technical Direction BTD2010/02 - Timber Bridge Design – Adoption of AS 1720.1:2010

C1-6 When is load rating required?

All existing bridges shall be assigned "as new" and “as is” load ratings.

Ratings of bridges are to be carried out in the following circumstances.

1. When ratings for the bridge in "as new" and/or "as is" conditions is not available.
2. Where there has been a recent change in condition of a bridge - such as damage by vehicle impact or where repairs have been carried out.

3. Where there is to be a change in the general traffic across the bridge or where a special load is to be operated.

Where rating of bridges in terms of fatigue is required, it shall be undertaken in accordance with AS 5100.

C1-7 Definitions

AASHTO	American Association of State and Highway Transportation Officials.
Alkali Aggregate Reaction	Reaction which occurs over time in concrete between the cement paste and aggregates. This reaction can cause expansion of the aggregate, leading to spalling and loss of strength of the concrete.
ANZRC	Australian and New Zealand Railway Code.
AREMA	American Railway Engineering and Maintenance-of-Way Association. Organisation which provides guidelines for practices for the design, construction and maintenance of railway infrastructure, which are requirements in the United States and Canada.
Assessor	Authorised CRN personnel undertaking Heavy Vehicle assessment on CRN assets.
CFF	Centrifugal Force Factors.
Compression	Force acting to compress a structural member.
Ductility	The ability of a solid material to deform under tensile stress. Practically, a ductile material is a material that can easily be stretched into a wire when pulled.
Dynamic Load Allowance (DLA)	A quantitative measure of dynamic effects exerted in addition to static loads by moving vehicles on highway bridges.
Engineering Load Rating	A rating using determined structural properties and loads in accordance with AS5100.7 by an engineer with engineering authority with load rating.
Extrados	The exterior (convex) curve of an arch or vault.
Flexural Strength	Strength of a structural member in bending.
Fracture Critical Member (FCM)	A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse. Tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function. fracture critical members (FCMs). An FCM is defined by the Code of Federal Regulations (23CFR650 – Bridges, Structures and Hydraulics) as “a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.”
GAV	General Access Vehicle.
Graphitisation	Formation of graphite free carbon in iron or low alloy steel, which occurs when their components are exposed to elevated temperatures over a long period.

	The formation of graphite is due to the nucleation and growth process that occurs when the steel is exposed to temperatures above 800°F (426°C).
HLP	Heavy Load Platform.
Heavy Vehicle	A Heavy Vehicle as classified by the National Heavy Vehicle Regulatory (NHVR).
Intrados	The interior curve of an arch or vault.
LLF	Live Load Factor.
Load Rating	Engineer with Engineering Authority for Load Rating.
MLMF	Multi Lane Modification Factor.
MTF	Multiple Track Factor.
Nick Bend test	A type of destructive testing that is used to evaluate the quality of a weld.
RF	Rating factor.
Shear Force	Force acting perpendicular to the axis of a structure component.
Ultimate Tensile Strength (UTS)	Often referred to a as ultimate strength, this is the maximum stress that a material can withstand while being stretched or pulled before failing or breaking.

Chapter 2 General principles

C2-1 Methodology

Load rating shall be carried out in accordance with AS 5100 “Bridge design” and other relevant Codes and Standards including AS/NZS 1170 “Structural design actions” and AS 4100 “Steel structures”.

C2-2 The Limit State concept

The limit state design principle requires that the assessed minimum capacity of the bridge must be greater than the assessed maximum loading by a defined margin of safety. As such, it is not a radically different process of design from traditional working-stress design methods but allows for the defined aim to be met in a logical manner.

Partial factors are individually defined and applied to elements of loading, structure, material and environment. The process then takes account of the constraints, according to the design life, and the performance limit states required.

C2-3 Limit State applied to rating

The rating process does not require any change in the design approach. The bridge is assessed against its ability to carry a repeated standardised live loading. Rating involves identifying the elements for which the partial factors in AS 5100 make allowance. For existing bridges, these elements may be capable of more accurate definition, resulting in a modification of the factors which would be used for design.

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 and superimposed dead load effects and parasitic, differential temperature and differential settlement effects.

The ability of a bridge to carry repeated general access live loads is assessed as a proportion of a nominated general access rating vehicle. Similarly, the ability of a bridge to carry a specific vehicle for a single pass or a small number of passes is assessed as a proportion of a nominated restricted access vehicle, operating under nominated conditions, e.g. speed restriction, location on bridge deck.

The rating procedure is carried out for all strength checks, e.g. moment, shear and the like, at all potentially critical sections, with the lowest rating factor determined being the Rating Factor for the bridge.

The general equation to determine the Rating Factor (RF) for bridges is therefore:

$$RF = \frac{\text{Available bridge capacity for live load effects}}{\text{Live load effects of nominated rating vehicle}}$$

C2-4 Rating validity

It must be recognised that an assigned bridge rating relates to an assessment carried out at a particular point in time (recording of the date that rating assessments are undertaken is therefore important). The “as new” rating of a structure indicates the maximum load capacity of a structure. The rating may reduce with time due to deterioration or overloading, or increase if strengthening is carried out. Therefore, a general rating capacity is assigned to a bridge for its "as new" and "as is" condition. The latter category allows for damage, deterioration or strengthening of the bridge. These factors will be taken into account in determining its load capacity.

Note that when providing a bridge rating it is essential that all significant conditions are also given, e.g. is rating for "as is" or "as new" condition? What surfacing depth applies? Is rating given for a particular speed restriction?

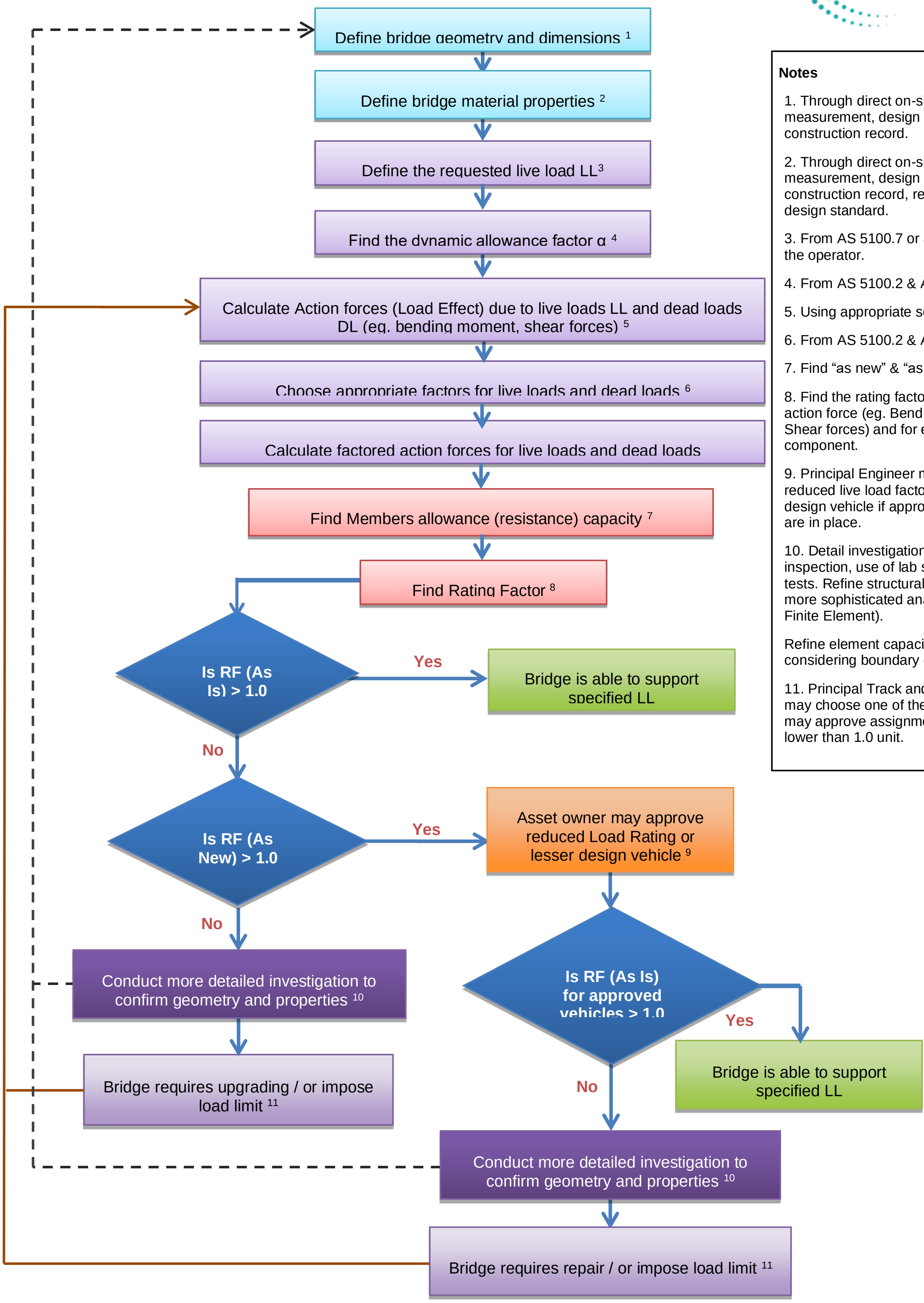
C2-5 Specific loadings

A bridge may also be rated for a specific live loading, i.e. abnormal loads or non-standard axle configurations. The same process is followed as for a general rating, but a load factor may be selected which reflects the variability and accuracy of load measurements in the particular case under consideration. Controls may be imposed to restrict the use of the bridge by that specific load, and load factors can be selected to reflect this. Controls may be imposed to restrict the use of the bridge for specific speed if the rating factor is found to be less than unity.

A separate procedure is adopted within the CRN for assessment of Heavy Vehicles on overbridges and is detailed in CRN CM 307.

C2-6 Rating procedure

The evaluation process follows a logical progression. The flowchart shown in Figure 1 indicates this process.



Notes

1. Through direct on-site measurement, design record, construction record.
2. Through direct on-site measurement, design record, construction record, relevant material design standard.
3. From AS 5100.7 or as specified by the operator.
4. From AS 5100.2 & AS 5100.7
5. Using appropriate software
6. From AS 5100.2 & AS 5100.7
7. Find “as new” & “as Is” capacities
8. Find the rating factor RF for each action force (eg. Bending moment, Shear forces) and for each component.
9. Principal Engineer may approve reduced live load factor or reduced design vehicle if appropriate controls are in place.
10. Detail investigation through site inspection, use of lab samples or NDT tests. Refine structural model, utilise more sophisticated analysis (e.g. Finite Element).

Refine element capacity by considering boundary condition.

11. Principal Track and Civil Engineer may choose one of these options or may approve assignment of RF as lower than 1.0 unit.

Figure 1 - Flowchart for rating process

C2-7 Statement of load rating

Determination of bridge live load capacity is generally based on assessment of superstructure capacity (Steel, Concrete and Timber trestles and columns are considered part of the superstructure components). The superstructures of some bridges, particularly masonry arch bridges, are sometimes found to have a high live load capacity. In order to account for the fact that substructure capacity generally cannot be determined (e.g. founding conditions are unknown) the maximum stated live load capacity shall be limited to "M1600 plus" (i.e. undetermined but greater than M1600) for Overbridges, and limited to "300LA plus" (i.e. undetermined but greater than 300LA) for Underbridges.

Note that any relevant parameters such as surfacing depth need to be included with the rating.

As outlined in Section C2-3, a Rating Factor shall be derived to indicate the theoretical load rating of a bridge element. This is in accordance with AS 5100.7.

A Rating Factor greater than or equal to 1 means that the bridge element under consideration can theoretically carry the nominated vehicle loading, based on the appropriate load factor on live load (γL).

Conversely, a number less than 1 means the bridge element does not theoretically have sufficient capacity based on the live load factor. This means that one of the following may then apply:

- Recommendation for restriction on the bridge for:
  - Allowing vehicle mass
  - Allowable vehicle speed
  - Number of lanes in service
- Acceptance of a reduced live load factor by the Principal Track and Civil Engineer. Taking into account factors such as the function of the structure, the level of traffic using the structure, and the age of the structure

C2-8 Rating Results

Rating results shall be expressed as the ratio of available member capacity to the applied load.

They shall be tabulated for “as new” and “as is”.

Vehicle types shall be shown.

The results of any fatigue analysis shall also be provided.

A typical layout for the presentation of the rating results is shown in Appendix 2.

Bridge component naming shall be in accordance with CRN Engineering Standard CRN CS 300 “Structures System”.

Notations shall be in accordance with AS 5100.

C2-9 Reporting

C2-9.1 General

A written report shall be prepared on the results of the load rating. The report is to include an executive summary at the front followed by:

- A statement regarding the particular Standards / Codes and other reference documents used in the rating
- A statement documenting and justifying the values adopted in the calculations including material properties and load factors
- Engineering details

- Appendices

The report shall include a general arrangement layout drawing of the bridge showing the arrangement of the main bridge components and the span layout.

Calculations and summaries shall be annotated in sufficient detail to clearly distinguish between the “as is” and the “as new” rating of individual components.

C2-9.2 Wrought iron test requirements

The reporting of test results for wrought iron structures shall include:

- Tensile properties
- Charpy values
- Origin of sample (i.e. name of location)
- Sample location size and orientation (e.g. transverse)
- Date of manufacture (or best estimate)
- Temperature (for Charpy tests)
- Extensometer charts (for tensile tests)

All test results shall be collated with existing records.

A mean minus 2 standard deviation value for yield and ultimate strength shall be used to determine the yield strength.

Chapter 3 Investigation and inspection

C3-1 Desktop study

Where possible, consult original records of design and construction to aid understanding of the bridge under consideration. However, this may not in itself be regarded as a substitute for an investigation of the bridge in its current condition. All cases will require detailed field inspection and measurement and in some cases this may extend to testing, e.g. material properties.

Similarly, the results of previous capacity investigations should also be reviewed (if available).

C3-2 Inspection procedure

A regular program of inspections is carried out on CRN bridges in accordance with the requirements of CRN Engineering Standard CRN CS 100 “Civil Technical Maintenance Plan” and CRN Engineering Manual CRN CM 302 “Structures Examination”. CRN Engineering Manual CRN CM 305 “Structures Assessment” outlines the action taken by UGLRL CRN personnel to certify structures after the examination process and includes a requirement to seek a review of the rating of a structure. Generally, the standard of routine inspections will be of a level sufficient to carry out a load rating if the procedures laid down are followed fully.

Study the examination report in detail before considering additional inspection.

Information in the examination report may assist in determining the most appropriate analytical model. UGLRL CRN Structures Examiners may be able to provide additional information to that given in the examination report which may assist in load rating.

C3-3 Measurement

C3-3.1 Geometric

It is important to be able to calculate section properties of members accurately. The actual size of components, geometric imperfections, and condition are necessary to determine these. Where this information is obtained by direct measurement, the general design-capacity reduction factors in AS 5100 may be increased. This is because the relevant sections of the code allow for some uncertainty when prescribing values for these factors for design situations. See AS 5100 for guidance on selection of factors for rating existing bridges.

For historic steel elements, section property information is available on the website and at the library of the Australian Steel Institute.

C3-3.2 Materials

Allowance may be made for changes in material properties if testing is carried out. A proper statistical assessment of results is required in order to derive characteristic properties. These shall comply with relevant Australian Standards. Note that material properties may increase with time, e.g. concrete strength gain or decrease e.g. decay of timber. Refer also to Appendix 1 for some information relating to iron and steel structures.

C3-3.3 Assumptions

If the examination report does not give the necessary measurements, and inspection to obtain measurements is not possible, or testing is not carried out, then an assumption may be made that components are in their "as-designed" condition, or "as constructed" condition if works as executed information is available. However, in this case, the design values for capacity reduction factors shall be applied. The same principle is also relevant to measurements of actual loadings and the application of load factors.

Note that if measurements have not been made to determine possible section losses etc. this must be clearly stated with the rating.

Chapter 4 Screening Assessment

C4-1 Introduction

Design and inspection documents (such as drawings) contain important information that is necessary for a thorough assessment of an existing structure. To enable a screening assessment to be performed for a converted rating factor, the following key information is required.

- **Age of Construction:** To enable assumption of design loading in the absence of drawings
- **Span dimension(s):** Used to obtain indicative design actions and member capacities
- **Type of Structure:** Transom Top/ Concrete deck – used for calculation of DLA
- **Last Inspection Date:** Reliability of current asset condition
- **Defects Reported:** Condition deterioration factors to be applied
- **Line Speed:** Used to refine DLA
- **Usage (loading)**

It shall be verified that the documents are correct to enable a converted qualitative rating to be derived for the bridge of interest.

C4-2 Standard design loading

The standard design loading of bridges under rail and traffic loads have changed over the years as design codes have developed. As a consequence of this change, historical design loads will need to be converted to reflect the current design code requirements.

C4-2.1 Underbridges

Based on Australian standard AS 5100.2, AS 5100.7, Austroads and AREMA, underbridges standard design loading as per construction era are as illustrated in Table 1.

Design Year	Standard Loading	Note
Pre 1974	Cooper E (Imperial)	AREMA, Iron and Steel Structures, Concrete Structures and Foundations.
1974 - 1996	Metric Cooper M	ANZRC Railway Bridge Design Manual
1996 - 2004	300-A-12	Australian Bridge Design Code (1996) – Railway Supplement
2004 to present	300LA	AS 5100 Bridge Design Code (2004)

Table 1 – Standard underbridge design loading per era

C4-2.2 Overbridges

Based on Australian Standards, AS5100.2, AS5100.7, Austroads and AASHTO, overbridges standard design load as per construction era are illustrated in Table 2.

Design Year	Standard Loading	Note
< 1931	Various	As per local authority guidance and requirements
1931 to 1948	M 18	Equivalent to AASHTO H20 Vehicle
1948-1976	MS18	Equivalent to AASHTO H20 Vehicle
1976 to 2004	T44	For bridge length of greater than 5.0m
2004 to present	SM1600	AS 5100 Bridge Design Code (2004)

Table 2 – Standard overbridge design loading per era

C4-3 Loads and lane factors

Due to the development of design codes over the years, there has also been a change in design philosophy from a working stress to Limit State approach. Load Ratings of existing bridges shall be undertaken using load and lane factors in accordance to AS5100.

C4-3.1 Underbridges

Rail load factors, dynamic load allowances (or impact factors) and multiple track factors used in different construction era are summarised in Table 3 – Live Load Factors, Dynamic Load Allowances & Multiple Track Factors below.

Design Year	Vehicle Type	LLF	DLA	MTF
<1974	Cooper E (Imperial)	TBA	TBA	TBA
1974-1996 ANZRC Railway Bridge Design Manual	Metric Cooper M250	1.0 (Working stress method)	Not included in Design Manual	For 1 track = 1.0 For 2 tracks = 1.0 For 3 tracks = 0.5 For 4 tracks = 0.25 For 5 or more tracks = as specified by the Engineer
1996-2004 Australian Bridge Design Code	300-A-12	1.7	For Bending moment For $L_\alpha \leq 3.6$m, $\alpha = 1.0$ For $3.6 < L_\alpha < 67$, $\alpha = \frac{2.16}{L_\alpha^{0.5} - 0.20} - 0.27$ For $L_\alpha \geq 67$ m, $\alpha = 0.0$ Shear, torsion and reaction to be 2/3 for the value for bending moment	For 1 track = 1.00 For 2 tracks = 1.00 For 3 tracks = 0.85 For 4 tracks = 0.70 For 5 or more tracks = 0.60
2004 to present AS 5100 Bridge Design Code	300LA	1.6	For Bending Moment for Ballasted Deck Spans For $L_\alpha \leq 3.6$m, $\alpha = 1.0$ For $L_\alpha > 3.6$m, $\alpha = \frac{2.16}{L_\alpha^{0.5} - 0.20} - 0.27$ For Bending Moment for open deck spans and spans with direct rail fixation For $L_\alpha \leq 2.0$m, $\alpha = 1.6$ For $L_\alpha > 3.6$m, $\alpha = \frac{2.16}{L_\alpha^{0.5} - 0.20} - 0.27$ Shear, torsion and reaction to be 2/3 for the value for bending moment	For 1 track = 1.00 For 2 tracks = 1.00 For 3 tracks = 0.85 For 4 tracks = 0.70 For 5 or more tracks = 0.60
	Reference 1974 A.N.Z.R.C Railway Bridge Design Manual – Chapter 1 – Loading for Railway Bridges			

Table 3 – Live Load Factors, Dynamic Load Allowances & Multiple Track Factors

In addition to these design loads, it is recognised that additional main line train loads are operational on the CRN network, these are noted in Table 4 below.

Designation	Train Load	Note
MF	Main Line Freight	Main line (82 Class) locomotives plus 100 tonne NHGF coal wagons
BF	Branch Line Freight	Branch line (422) locomotives plus 81 tonne NGTY wheat wagons
LB	Light Branch Line Freight	Branch line (48) locomotives plus 76 tonne NGTY wheat wagons
XP	XPT/ eXplorer	Passenger train
SB92	Short Bogie	String of 11 metre bogie wagons such as RCGF steel coil wagons and a number of open wagons used in ore transport

Table 4 – Additional Main Line train loads

In addition to 300LA the following loads shall be used in the rating:

- Class 1 & 2 Lines – MF
- Class 3 lines – BF
- Class 5 – LB

C4-3.2 Overbridges

Traffic load factors, dynamic load allowances (or impact factors) and multiple track factors used in different construction eras are summarised in Table 5 below.

Vehicle Type	LLF	DLA	MLMF
M18 & MS18	1.3[*6]	$(1.0+\frac{50}{L+125}) \leq 1.3$ [*6], L is the span length in ft	For 1 & 2 lanes = 1.0[*8] For 3 lanes = 0.9 For 4 lanes or more = 0.75
T44	2.0[*1]	1.2 [*2]	For 1 lane =1 [*3] For 2 lanes = 0.9 per lane For 3 lanes =0.8 per lane For 4 lanes = 0.7 per lane For 5 lanes = 0.6 per lane For 6 lanes or more =0.55 per lane
SM1600	1.8	1.3	For 1st lane =1 For 2nd lane = 0.8 For 3rd and additional lanes = 0.4
*[*1] As per AS5100.7* *[*2] As per AS5100.7, consider the minimum value of 1.2 for conservative analysis* *[*3] As per AS5100.7,* *[*6] As per ASSHTO HS loading* *[*8] As per AS5100.7,*			

Table 5 – Load and lane factors, DLA and multiple lane factors for different road vehicle types

C4-4 Assessment Process

The flow chart Figure 1 illustrates the load assessment process to be followed.

C4-4.1 Age of construction

The age of construction can give an indication of the design loads that were employed. This then enables a converted rating factor to be derived. The age of the structure, together with existing rail and vehicular traffic loads, provides a basis to assess structures in relation to previous performance.

C4-4.2 Span dimension(s)

This information is used to evaluate indicative loads as well as section capacities to be used for the rating factor.

C4-4.3 Type of structure

The type of underbridge structure known as the characteristic length (Lα) for each component is dependent on the structural geometry. The characteristic length is used for the evaluation of the dynamic load allowance for rail loadings.

For the assessment of overbridge structures, the structural geometry is required to refine the Dynamic Load Allowance for the T44 design vehicle.

C4-4.4 Last date of inspection

The last date of inspection provides information to assess the reliability of the current condition of the structure and hence the validity of rating the structure using this process. A collection of previous inspection reports may also give an indication of any accelerated deterioration of structural components.

C4-4.5 Defects reported

The defects report may allow appropriate condition factors (or loss of section) to be applied to known deteriorated structural members to allow a converted rating to be evaluated. Refer to section C9-1 Section loss and loss of fasteners in steel members for qualitative estimates of loss of section.

C4-4.6 Line speed

The known line speed can be used to refine and adjust the DLA to enable a converted rating to be evaluated for underbridges.

C4-5 Screening assessment results

When the above structure information is available and confirmed reliable, a screening assessment can be performed to convert the old rating factor to a converted rating based on an assumed design or operational load. The level of analysis (i.e. one dimensional) must be recorded in the assessment.

For conversion of rating factors from Working Stresses to Limit State and different design/operational loads, the following equation shall be used.

$$RF = \frac{\emptyset(Sold\ design \times (1+\alpha old\ design)) \times S.F}{\gamma LL,new \times Snew\ design \times (1+\alpha new\ design)}$$

For conversation of rating factors between Limit State design and different design/operational loads, the following equation shall be used.

$$RF = \frac{\gamma LL,old \times Sold\ design \times (1+\alpha old\ design)}{\gamma LL,new \times Snew\ design \times (1+\alpha new\ design)}$$

Where: (from AS5100)

$\emptyset$ = Capacity Reduction factor in Limit State design

$S.F$ = Safety Factor used in Working Stress design

$Sold\ design$ = Design action of old load

$Snew\ design$ = Design action of new design load or operational load γLL,old = load factor of old design load

$\gamma LL,new$ = load factor of new design load or operational load

$\alpha old\ design$ = Dynamic load allowance of old design load

$\alpha new\ design$ = Dynamic load allowance of new design load or operational load

If the Rating Factor is less than one, further restrictions may need to be imposed. These need to be approved by the Principal Track and Civil Engineer.

Chapter 5 Bridges without a formal load rating or design information

Bridges without a formal load rating or design information may be assessed under the provisions of ISO 13822, Section 8, Assessment based on satisfactory past performance, provided the following conditions are satisfied.

- The historical bridge inspection and assessment reports do not reveal any evidence of significant damage, distress or deterioration of the structure or its critical elements
- The number of inspection reports meets the frequency required as stipulated in CRN CS 100 Civil Technical Maintenance Plan
- Review of the structural system, including investigation of critical details and checking them for stress transfer is to the satisfaction of the Structures Superintendent
- The Structures Superintendent is satisfied that the structure has demonstrated satisfactory performance with extreme actions due to use and environmental effects which may have occurred
- The Structures Superintendent deems that the predicted deterioration, taking into account the present condition and planned maintenance ensures sufficient durability
- The Structures Superintendent is satisfied that there have been no changes that could increase the actions on the structure or affect its durability, and no such changes are anticipated
- Following a review of the historical bridge inspection and assessment reports, the Structures Superintendent, at their discretion, may impose restrictions on the bridge whilst a formal load rating assessment is being undertaken

Chapter 6 Formal load rating procedure when no design information is available

Figure 2 provides a flow chart of the formal load rating process for a bridge that has limited or no design, as constructed or updated drawings.

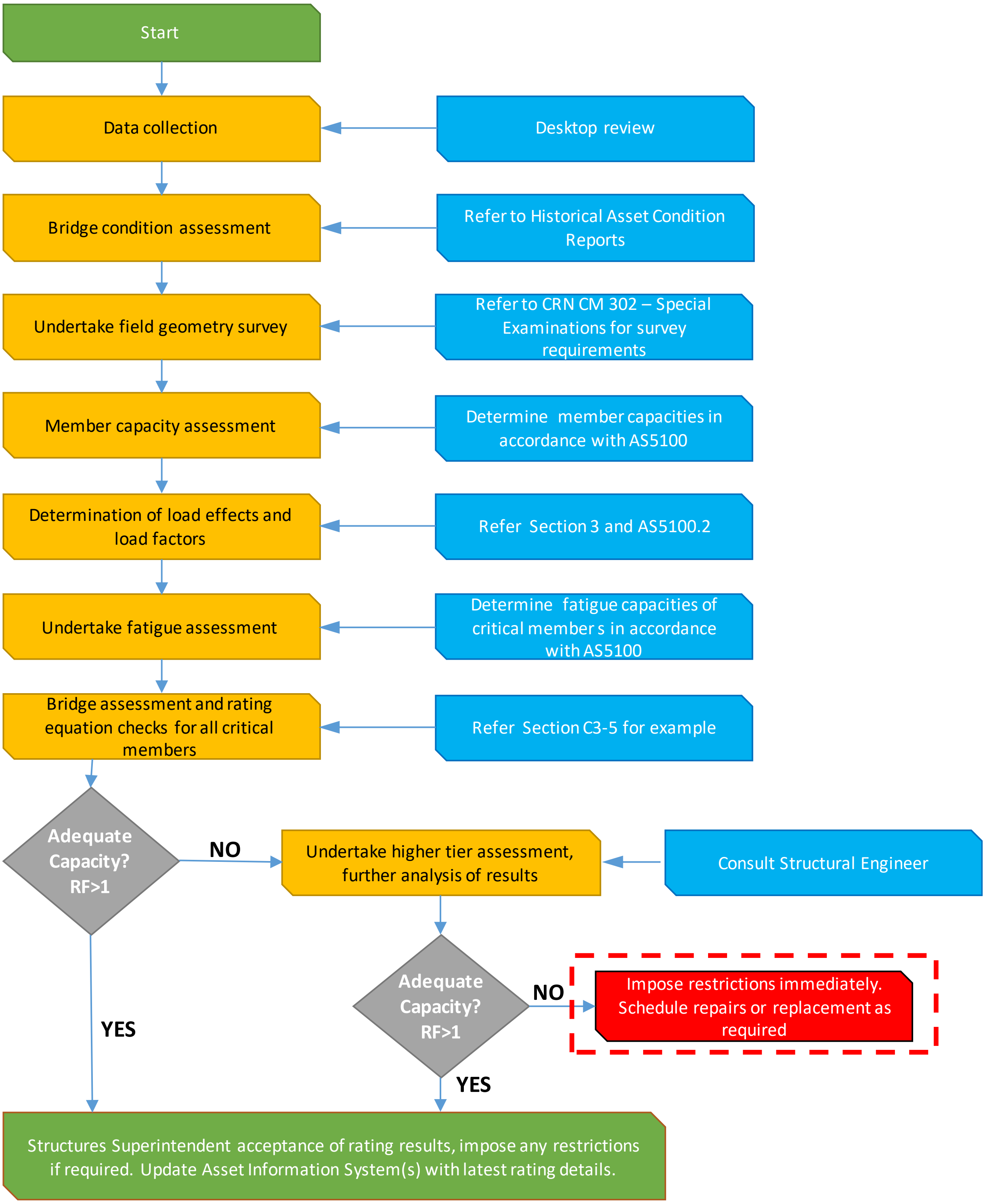


Figure 2 – Flow chart of load rating process when no structural information is immediately available

C6-1.1 Data collection

Table 6 contains information required, its relevance and criticality to the bridge load rating process.

Information	Relevance	Criticality	Requirement
Age of structure	Historical loading and past performance	High	Mandatory
Span length of structure	Determine load effects	High	Mandatory
Type of structure	Calculation of DLA	High	Mandatory
Last inspection report	Confirmation of structural integrity	Medium/ High	Mandatory
Speed of vehicles traversing the structure	Refinement of DLA	High	Mandatory
Live loadings	Determine load effects	High	Mandatory
Design drawings	Design loading	Medium/ High	Where available
Material properties	Determine section capacities	Medium/ High	Mandatory

Table 6 – Bridge information requirements

Some of the material properties required to determine section capacities are, but are not limited to, the following.

- The minimum yield strength of structural steel.
- The compressive strength of concrete
- The minimum yield strength of reinforcing steel
- The type e.g. stress relieved (SR), normal relaxation (NR) or low relaxation (LR) and tensile strength of pre-stressing steel

Where material properties are unknown, they shall be determined using one of the following methods.

In the absence of any design documentation, an approximation of the age of the bridge shall be made and conservative material properties and loading criteria shall be adopted.

- **Known Age:** when the construction age of the bridge is known, then an assumption can be made of the design load, and the bridge will have assigned an assumed load limit
- **Estimated Age:** when the construction age of the bridge can be estimated (i.e. using measurements material testing, comparison to old standards), then an assumption can be made of the design load, and the bridge will have assigned an estimated load limit

Additionally, in order to gain a better estimation of the actual material properties of the bridge components, the Principal Track and Civil Engineer may deem, if necessary, to carry out non-destructive testing or sampling of the critical bridge elements when the field geometry survey is undertaken.

C6-1.2 Bridge condition assessment

The condition of the bridge shall be assessed to assist in determining the current capacity of the components of the structure and its foundations.

No rating of the bridge shall be considered valid until a special assessment and / or detailed assessment has been undertaken to determine the current condition of the bridge and the extent to which the condition affects the load-carrying capacity or general safety of the bridge.

The bridge shall be assessed in its current condition i.e. its capacity will be assessed "as-is" rather than its "as-new" condition.

A bridge that has a primary structural element with defects that affect its functionality as designed and requires action shall be rated based on a theoretical design alone. The capacity shall be determined based on a rational engineering assessment.

C6-1.3 Undertake field geometry survey

Refer to CRN CM 302 Special Examinations, for a list of structurally critical members in bridges whose dimensions need to be accurately measured during the field geometry survey.

In order to accurately assess the structural capacity of the bridge the actual current geometry, dimensions and section properties of the bridge and its components, including the foundations shall be measured directly. The assessment of structural resistance shall allow for all geometric imperfections and eccentricities caused by inaccurate construction damage or any other cause.

Assessments of section properties shall consider:

- The actual size of the member and internal components including any variations caused by corrosion
- Other deterioration causing loss of section, such as wear
- The uncertainties of the position of internal components, such as prestressed and non-prestressed components

C6-1.4 Member capacity assessment

Critical sections to be analysed shall include, but not be limited to:

- Locations of maximum moments
- Sections adjacent to the supports for negative moments
- Locations of maximum shear and/or torsion
- All regions of curtailed reinforcement or changes in reinforcement profile
- Changes in section
- Connections

Where relevant, assessments of stability shall also be carried out.

C6-1.5 Determination of load effects and load factors

The determination of load effects and load factors shall be in accordance with Chapter 4 and AS5100.2.

C6-1.6 Fatigue assessment

Assessment of bridges for fatigue shall consist of determining the cumulative fatigue damage of the critical details or components of a bridge, and of determining the nominal fatigue life of the bridge. The assessment shall be undertaken by using the procedures for fatigue specified in AS/NZS 5100.6, together with other relevant information. For the purposes of assessment, the cumulative fatigue damage shall be the sum of the damage due to historical loading. The nominal fatigue life shall be considered to have been reached when the cumulative damage sums to unity.

In assessing a bridge for fatigue, actual strains at critical details may be measured to deduce stresses. The stress pattern due to a defined load shall be assessed to determine the effective number of load cycles applied to the structure, or the detail being considered, by the passage of one loading sequence.

The effect of worn wheels may increase the number of cycles, the amplitude and rate of strain for railway bridges. The frequency of worn wheels shall be considered.

If a bridge has reached its theoretical fatigue life, a risk management strategy shall be put in place which may include reducing the load limit, reducing speed limit on the bridge, monitoring fatigue cracks and/or replacement of the bridge or the fatigued components.

When assessing a road bridge, an assessment of the actual loads and related number of stress cycles shall be made in accordance with AS 5100.2.

When assessing a bridge, the actual loads shall be considered. The effective number of load cycles (n) specified in AS 5100.2 shall only be used if the assumptions detailed in AS 5100.2 Supp. 1 are known to be appropriate.

C6-1.7 Special criteria for rivets and bolts

For the purposes of fatigue calculations, tight rivets in mechanically fastened connections may be treated as bolts of Category 8.8/TF. Connections with loose rivets, or connections that are made of bolts not tightened in accordance with the requirements for Category 8.8/TF, shall be assigned a detail Category 50 as defined in AS/NZS 5100.6.

C6-1.8 Bridge assessment and rating equation checks for all critical members

The determination of the load rating factor of a bridge shall be carried out by comparing the factored live load effects of the nominated rating vehicle with the factored strength of the bridge after subtracting the load effects from the factored permanent loads including dead and superimposed dead load effects, parasitic, shrinkage, creep, bearing, friction, differential settlement and temperature effects.

The load rating of a bridge shall be carried out for all strength checks (e.g. moment, shear, torsion and the like) at all potentially critical sections as described but not limited to those listed in CRN CM 302, with the lowest rating factor determined being the rating factor for the bridge.

For the purpose of rating, the general strength equation for bridges shall be calculated from the following equation.

Single load effect

$$\varphi R_u \geq \gamma_g S_g^* + \gamma_{gs} S_{gs}^* + S_p^* + S_s^* + S_t^* + \gamma_L (1+\alpha) W(S_L^*)$$

Therefore, the rating factor (RF) for bridges shall be calculated from the following equation:

$$RF \leq \frac{\varphi R_u - \left(\gamma_g S_g^* + \gamma_{gs} S_{gs}^* + S_p^* + S_s^* + S_t^*\right)}{\gamma_L (1+\alpha) W(S_L^*)}$$

For conversation of rating factors between Limit State design and different design/operational loads, the following equation shall be used:

$$\mathrm{RF} = \frac{\text{Available bridge capacity for live load effects}}{\text{Live load effects of nominated rating vehicle}}$$

where

φ = capacity reduction factor

Ru = calculated ultimate capacity

γg = load factor for dead load

$Sg*$= load effects due to dead load

γgs = load factor for the superimposed dead load

$Sgs*$= load effects due to superimposed dead load

$Sp*$ = load effects due to parasitic effects or prestress

$Ss*$ = load effects due to shrinkage, creep, differential settlement and bearing friction

RF = rating factor

$SL*$ = load effects due to the live load used for the assessment

W = a factor representing:

- MTF for railway traffic bridges, that is, the multiple track factor determined in accordance with AS 5100.2
- ΣALF for road traffic bridges, that is, the accompanying lane factor determined in accordance with AS5100.2

NOTE: The ΣALF effect is the sum of load effects of each loaded lane with the relevant ALF.

α = dynamic load allowance

LR = rated load

LRV = nominated rating vehicle.

Combined actions

The interaction equation for combined actions shall be as given in AS 5100.5 for concrete, AS/NZS 5100.6 for steel and concrete and AS1720 for timber.

Where the rating for a specific bridge is assessed as being less than required (i.e. RF <1) the subject bridge shall be deemed to not satisfy the nominated vehicle or loading. The Structures Superintendent shall impose short term restrictions on the bridge whilst a higher tier assessment is undertaken by a Structural Engineer.

Following the outcome of the higher tier assessment, should the rating for a specific bridge be assessed as being adequate (i.e. RF ≥ 1) the full load rating report shall be referred to the Structures Superintendent who shall make the final bridge assessment, rating and impose restrictions on the subject bridge if required.

Should the outcome of the higher tier assessment yield a rating less than required, the Structures Superintendent shall impose permanent restrictions on the subject bridge and schedule in repairs or replacement of the defective bridge components as highlighted in the higher tier assessment.

Chapter 7 Triggers for load rating

Although bridges which contain ‘as-built’ loading information or have sufficient structural information may enable a converted rating to be calculated, it is important to realise what the limitations of this process are, and when further works need to be undertaken to enable a full load rating to be performed. This section of the manual is intended to provide some guidelines as to when a load rating should be undertaken, as well as providing some guidance to examiners to escalate defects for an engineering-load rating assessment. This means that the deterioration and defects are significant enough to warrant a re-rating to be established and without intervention. Failure of the component may be expected in the short to medium term. Refer to CRN CM 302 C4-4 for guidelines of known defects that warrant a review of load rating.

C7-1 Section loss and loss of fasteners in steel members

Section loss in primary structural members (such as girders and connections) directly affects the load-carrying capacity of the structure. Where condition reports or inspection reports identifies significant section loss in members (i.e. loss of section in the tension flange of a steel member), a load rating assessment should be undertaken to recalculate the section capacity for a revised load rating. Similarly, where fasteners such as bolts or rivets are missing in critical connection details (i.e. gusset plates or beam splices), recalculation of the capacity will also be required. A load rating should be triggered if:

- Severe surface corrosion in steel members or connection members resulting in substantial loss of section are detected
- Welds are cracked
- Fasteners are severely corroded with loss of section. Some may be loose or missing, allowing extensive movement
- Girder/Beam exhibits residual out-of-plane deformations
- Cable/Hanger: Hangers may be sliding along cables. The cables may have slackened noticeably. Cable anchorages are severely cracked or have moved or slipped. Cables may be severely abraded with a number of broken strands

		
Significant cracking of weld Section could potentially fail	Severely corroded gusset plate	Severely corroded steel section

Figure 3 – Section loss in members

C7-2 Cracking and spalling of concrete and corrosion of reinforcement

Severe cracking and spalling of concrete in the structure may indicate either damage to the structure or significant corrosion in the reinforcing steel or prestressed strands. A detailed inspection should be undertaken to establish the extent of the damage and the extent of the corrosion for a revised load rating. Load rating should be undertaken if the following defects are noted:

- Heavy cracking (>0.7mm width) with fretting and spalling possibly present, severe corrosion of the reinforcement over large areas, resulting in substantial loss of section. Prestressing strands (where present) may be broken of exhibit signs of advanced corrosion
- Deck may have extensive longitudinal cracking with differential movement between sections
- A pattern of tension cracks may be present with medium cracking (>0.3 & ≤0.7mm width)

		
Severe impact damage with total destruction of the beam Immediate action required	Substantial loss of section, spalling of concrete	Substantial loss of reinforcement section and failure of beams Note the absence of discolouration/ staining

Figure 4 – Examples of cracking and corrosion of concrete

C7-3 Bridge strengthening/refurbishments

On structures where significant structural changes have been implemented, a load rating could be triggered. Items such as a deck overlay, addition of a heavier railing, replacement of timber sleepers to concrete sleepers, beam repairs, new beams, widening of the structure, or significant substructure repair or alterations could trigger a load rating. Additionally, the assessor must be aware of any significant changes in dead load as a result of the works performed on the bridge as well as the revised section capacity due to the strengthening or repair works undertaken.

Where the partial configuration of the structure has occurred (i.e. replacement of a failed girder with a newer design), this may also trigger the need for a formal load rating.

C7-4 Differential settlement

Depending on the type of structural system in place, signs of differential settlement at the supports may require a load rating to be triggered. A simply supported structure will need visual investigations to check whether track lines are within operational tolerances. Differential settlement of supports on continuous structures will create additional member stresses hence a load rating will be required to assess this effect.

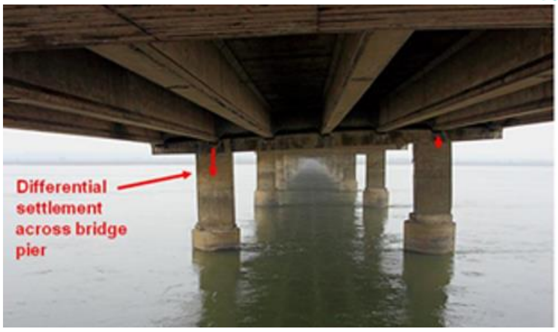
	
Settlement of Pier resulting in beam sag	Differential settlement of one column in a pier due to earthquake

Figure 5 – Differential settlement in piers

C7-5 Post incident

All bridges shall be subject to a visual inspection after major accidents, flood, earthquake, bushfires or other incidents e.g. impact due to collision. Photographic examples of the effects of such events on bridges and bridge components are shown below.

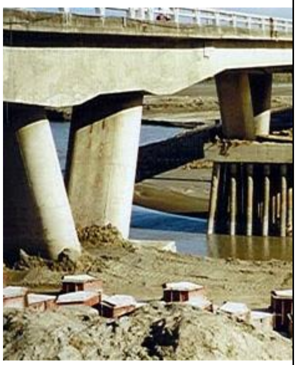
		
Bridge damage due to fire Immediate action required	Major scour to bridge pile cap due to large flood event Immediate action required	Distortion of main girder due to impact to bottom flange Immediate action required

Figure 6 – Effects of Incidents on bridges and bridge components

1.1.1 C7-5.1 Fire or explosion events

Bushfires and explosions can cause significant damage to all bridges and bridge element types. Timber bridges are the most susceptible to fire damage as once heated above a certain temperature they will readily combust.

During an inspection, items that can affect the load-carrying capacity of a bridge or bridge elements and must be assessed include:

- The extent of the heat-affected zone in concrete and timber components using NDT and/or intrusive methods e.g. drilling
- The extent of cracking – specifically, the width and depth in concrete and masonry or brick structures
- The extent of heat damage/deformation and change in material properties e.g. warping of steel beams causing a reduced stiffness

- Assessment of prestressing tendons and other high-strength steel components that are susceptible to heat damage such as Carbon Fibre Reinforced Polymer (CFRP) and bonding materials
- Assessment of displacement (absolute and relative) of bearings and superstructures
- Examination of colour change in concrete components

C7-5.2 Flood events

A special inspection of all structures is required following a major flood event. This investigation should assess/investigate the extent of:

- Damage to piers, abutments and bridge superstructures from debris impact (floating trees, vehicles and vessels for example)
- Lateral movement or uplift of bridge superstructures and bearings due to debris loading and buoyancy effects
- Scour of river bed under and adjacent to foundations, which may not be evident after the flood-waters subside
- Aggradation of river bed adjacent to foundations and superstructures
- Damage to approach embankments and beaching

Note that excess pore pressure and draw-down effects can cause failure, rotation or settlement of abutments, retaining walls, other structures and river-banks.

C7-5.3 Earthquake events

All bridges may suffer severe vertical and or lateral accelerations and movements due to earthquakes. Bridges can suffer bearing displacement, closure or opening of gaps between adjacent components causing spalling or failure of concrete members, distortion or tearing of steel members, damage to expansion joints, settlement and rotation of foundations, piers and abutments due to soil liquefaction.

Retaining walls and other structures reliant on soil-structure interaction could become unstable and suffer damage or failure of main components as a result of settlement, rotation or collapse.

Masonry culverts, arches and retaining walls and other non-ductile structures are likely to suffer severe cracking of masonry and mortar joints, settlement of foundations, rotation of approach walls and settlement of the contained roadway.

C7-5.4 Post impact and vandalism

Bridges are susceptible to impact damage due to errant vehicles and derailed trains. In the event that a main structural member is damaged as a result of impact or vandalism, temporary restrictions or the closure of the bridge shall be imposed until a formal load rating can be undertaken or repair works carried out to restore the bridge to its full load-carrying capacity. When undertaking a load rating on a bridge that contains a damaged structural member, a reduced member stiffness shall be employed when deriving the member's structural capacity. Additionally, the reduced stiffness shall be employed in all subsequent modelling of the bridge to account for the additional actions that will be introduced into the adjacent, stiffer, structural members.

During the time of this investigation, the side of the bridge that has been impacted shall be closed to all traffic.

Chapter 8 Overbridge loadings

C8-1 Changes in design loads

Over the years, design loads have changed as design codes have developed. Overbridge design loads have been expressed as:

- M18/MS18
- T44
- HLP320 / HLP400
- SM1600

Ratings for CRN overbridges are generally expressed as ST42.5, BD62.5, T44 or M1600 loadings.

Most bridges have been designed to older design codes and do not necessarily comply with the current design code.

Details of the changes in loadings are given below.

C8-1.1 1970 Highway Bridge Design Specification (metric version, 1973)

Two systems of loadings were specified M loadings, consisting of a 2-axle truck and MS loading consisting of a tractor truck with semi-trailer. Bridges in Metropolitan areas and on main roads and highways were designed for MS18-44 loading. MS loading was heavier than the corresponding M loading.

Further guidance regarding the application of these vehicles can be found in Section A2.4 of AS5100.7.

The standard MS Truck is as shown below.

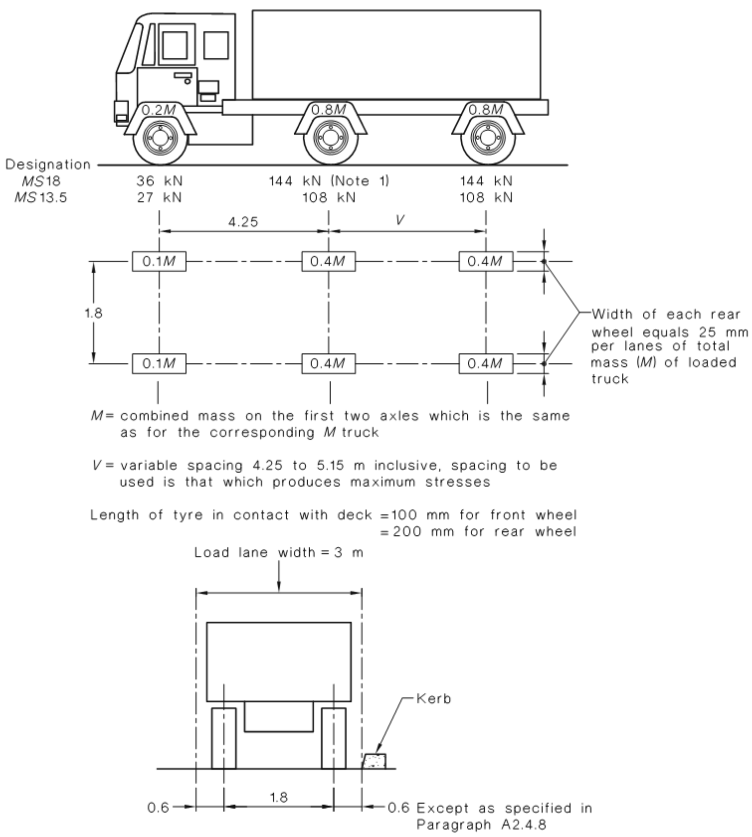


Figure 7 - MS Truck

C8-1.2 1976 NAASRA – Design Live Load

The live load consists of the weight of the applied moving load, such as the standard vehicle load A14 or T44, the standard abnormal or special abnormal vehicle load and the walkway load, where applicable.

Further guidance regarding the application of these vehicles can be found in Section A2.3 of AS5100.7

The standard T44 truck is as shown below.

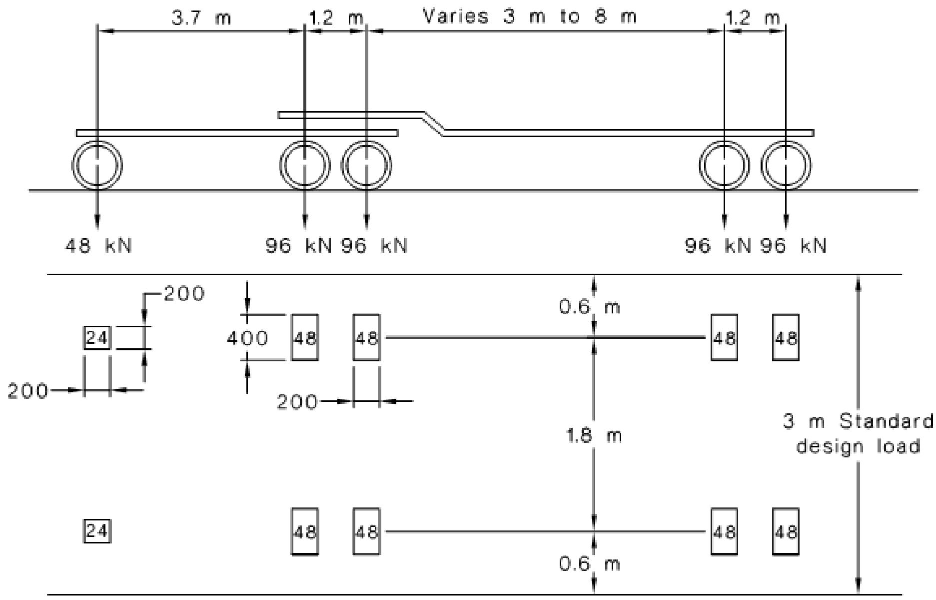


Figure 8 – T44 Truck loading (1976)

C8-1.3 1992 Austroads Bridge Design Code and 1996 HB 77.2—Design live load

Live loads consist of T44 Truck loading, identical to the 1976 NAASRA design live load, plus Heavy Load Platforms, if required by the road authority. For the T44 truck load and L44 lane load, the calculation of Dynamic Load Allowance was based on the first flexural frequency of the superstructure.

Further guidance regarding the application of these vehicles can be found in Section A2.2 of AS5100.7.

The HLP truck is as shown below.

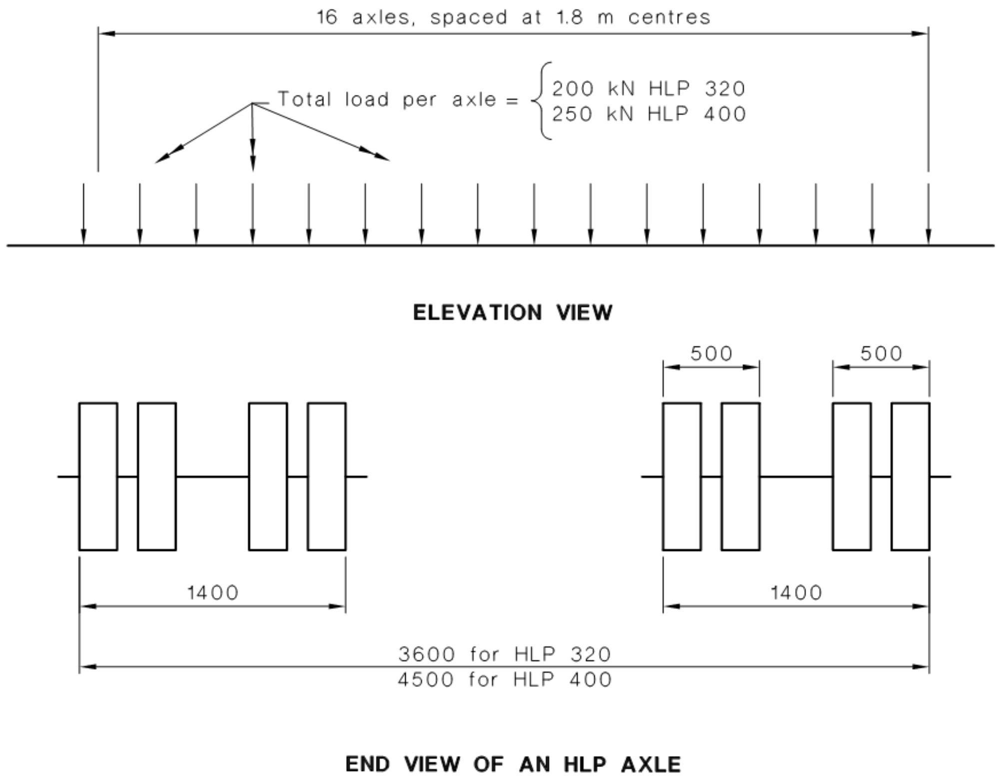


Figure 9 – HLP Vehicles

C8-1.4 AS 5100 Bridge Design code (2004 / 2017)

SM1600 represents a series of design loads – W80 wheel load, A160 Axle Load, M1600 moving vehicle load and S1600 Stationary Traffic Load. HLP320 or HLP400 may also be applied if requested by the road authority.

For further details regarding the application of Live Loads under AS5100, refer to Section 6 of AS5100.2.

When looking at global effects, the M1600 moving vehicle load usually controls the load rating of the structure.

Figure 10 below shows the M1600 Moving Traffic Load.

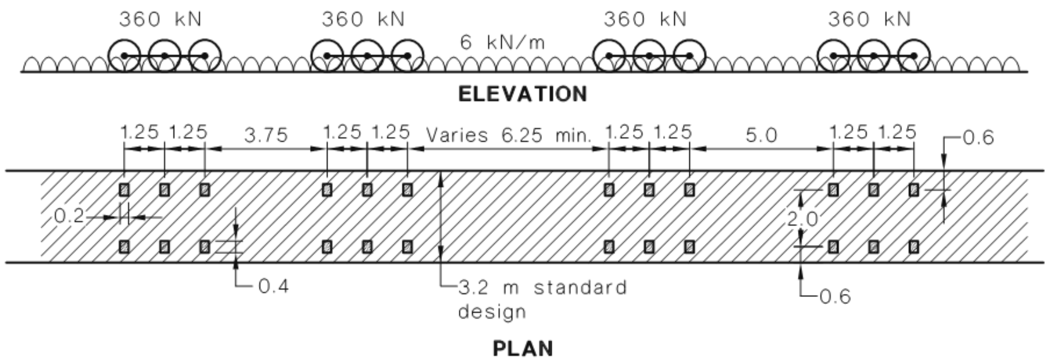


Figure 10 – M1600 Moving Traffic Load

Load ratings of bridges are now to be related to the T44 and ST42.5 loadings. Computation is performed for every critical structural element with the load capacity being determined as a proportion of the T44 and ST42.5 loadings loading.

Irrespective of the code or standard referred to, the higher the number the stronger the bridge i.e. it can carry higher loads and has more ability to withstand the effects of any defects.

Section 5 of CRN CS 320 covers the design criteria for Overbridges on the CRN network.

C8-2 Rating Vehicles

As a minimum, Overbridges shall be load rated for M1600, T44, MS18, ST42.5 and BD62.5 vehicles.

In addition, the Principal Track and Civil Engineer, in consultation with the road authority and other stakeholders, may elect to nominate additional design vehicles including HLP320 and HLP400. Further design vehicles may also be nominated, as outlined below.

C8-2.1 Specific live loadings

The design vehicles set out in Section C8-1 above are not fully representative of typical road vehicles.

Further to the T44 and M1600 rating vehicles, the Principal Track and Civil Engineer may elect to nominate further specific-design vehicles, representative of vehicles that can use the road network without special permits or permissions, such as 42.5t Semi Trailers or 62.5t B-Doubles.

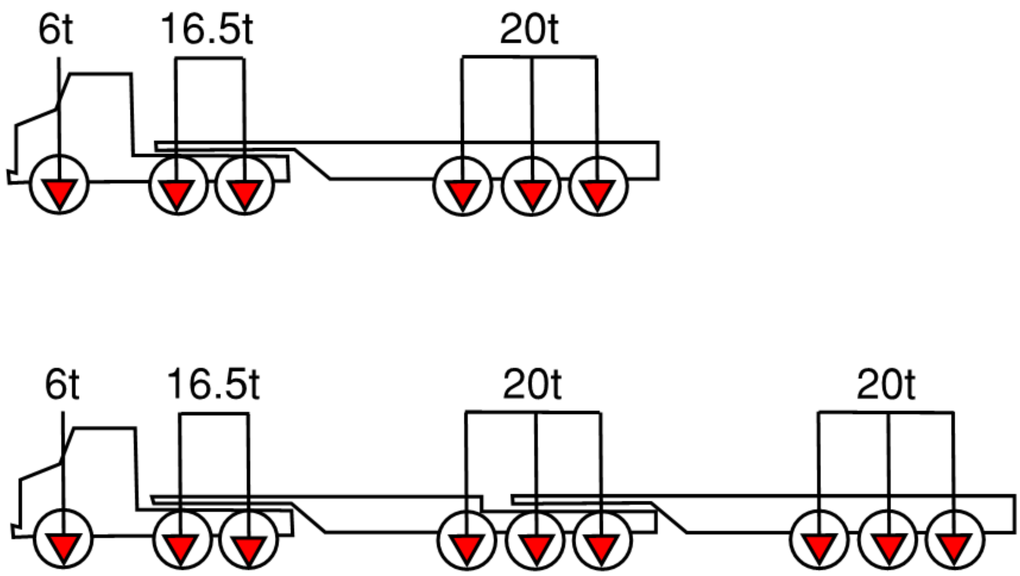


Figure 11 – Semi-trailer and B-Double

Further to generic truck loads, bridges may also be load rated for specific oversized vehicles for specific applications, such as the transport of very large, indivisible loads, such as power station transformers or large mine equipment.

If it is required to assess a bridge against a specific live loading, the procedure is the same as that depicted in the preceding sections of this manual.

Note that the Track and Civil Principal Engineer may approve reduction of load factors for specific live loads with the bridge capacity being, in effect, increased for the specific live load. Direct comparison of the load effects of the specific live load with the bridge rating for general traffic may therefore not be appropriate.

C8-3 Choice of load factors

C8-3.1 General

Rating shall generally be undertaken using the loads and load factors in accordance with AS 5100. For specific live load vehicles, load factors should be used which are most similar to the standard vehicles. For example, T44 load factors and dynamic load allowances should be used for general access vehicles (where there may be a higher risk of overloading), and HLP load factors should be used for specific oversize, over-mass vehicles.

Assigned load factors for load rating of existing bridges are based on the degree to which actual loadings are measured for a particular bridge. Dead and Superimposed Dead Load can be relatively easily and accurately estimated. Particular notice must be taken of the position and effects of services which have been added during the life of the bridge. In view of this, the Principal Track and Civil Engineer may approve reduction of dead load factors from the values used for design of new bridges.

General live loading is less predictable such as the possibility of overloaded trailers. Therefore the rating live load factor must be much the same as that for design, except in the case of a Specific Live Loading (see sectionC8-2.1).

C8-3.2 Load factors

Load factors for dead loads, superimposed dead loads and live loads shall be in accordance with AS 5100.

Where the load carrying capacity rating of a component or connection is less than unity (1.0), the reduced load factor for Live Load (LL) shall be reported based on rating being equal to unity (1.0).

For example, if rating = 0.8 with LL load factor = 2.0, then LL load factor will be less than 1.6 for rating = 1.0.

The Principal Track and Civil Engineer shall determine if a load factor lower than the AS 5100 value is acceptable.

C8-4 Dynamic load allowance

The dynamic load allowance (DLA) specified in AS 5100 shall be used in the assessment of railway bridges.

C8-5 Nosing load

The Noise load specified in AS 5100 shall be used in the assessment of bridges.

C8-6 Wind load

The Wind load specified in AS 5100 shall be used in the assessment of bridges.

C8-7 Overbridges load restrictions

To ensure the safe use of maintained overbridges, it is important to advise road users and authorities of the limitations of the bridge capacity to support the passing of the General Access Vehicles (GAV) Semitrailer ST42.5. This is typically in the form of sign posting the bridge with load limit sign as per approved RMS load limit signage system. There are two approved RMS load limit signs, these are R6-3 and R6-17 as shown in Figure 12 below.

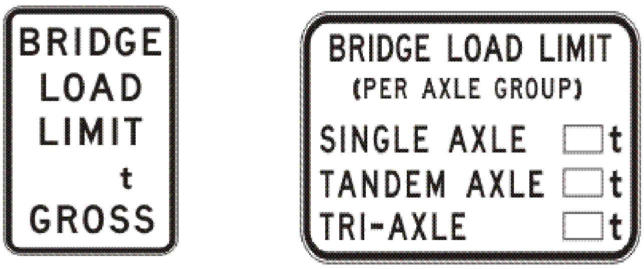


Figure 12 – RMS load limit sign R6-3 (left) and RMS load limit sign R6-17 (right)

If an overbridge is found to have a Rating Factor (RF) less than unity (1.0) for GAV Semitrailer (42.5 tonne and length <12.5m) and/or GAV B-double (50 tonnes and length <19m), the assessor should provide details for the two RMS approved load limit signs and advise on the most suitable option for the bridge scenario.

C8-7.1 RMS sign R6-3

- If RF for Semitrailer ST42.5 <1.0; but RF for B-Double 50t >1.0

Gross Limit = RF_ Semitrailer x 42.5t

- If RF for Semitrailer ST42.5 >1.0; but RF for B-Double 50t <1.0

Gross Limit = RF_ B-Double x 50t

- If RF for Semitrailer ST42.5 <1.0; and RF for B-Double 50t <1.0

Gross Limit = Less of (RF_ Semitrailer x 42.5t; RF_ B-Double x 50t)

C8-7.2 RMS sign R6-17

The sign posted values (allowable limit) for Single, Tandem, and Tri-axle groups shouldn't exceed the maximum mass limit Single, Tandem, and Tri-axle groups on GAV vehicles as per Table 7.

Axle Group	Maximum Mass limit (tonne)
Single axle (group) fitted with single tyres	6
Tandem axle group fitted with dual tyres	16.5
Tri-axle group fitted with dual tyre	20

Table 7 – General Access Vehicle Mass limit for Truck type vehicle

The assessor shall find the allowable limit for each axle group and define the controlling axle groups. It is expected that the single axle group will not govern the bridge capacity and usually it doesn't require any reduction unless otherwise advise by the assessor.

The calculation of the allowable limit for each axle group can be done through the re-model of the governing GAV vehicle(s) with different values of the axle group mass limit to find the allowable axle limit that achieve a Rating Factor above unity.

The allowable limit for each axle group shall be calculated by the assessor in such a way that applying the allowable limit for the specific axle group in the presence of the adjacent axle group(s), result in a Rating Factor of at least a unity.

As axle groups are connected to each other through the frame of the trailer, reducing the mass on one axle group influences the mass on the adjacent axle group(s) as the trailer mass will be re-distributed between axle groups. The single axle group is carrying the prime mover load and normally it will not share the trailer load, thus usually no reduction is required for the Single axle group, unless otherwise advise by the assessor.

It is not appropriate nor accepted to calculate the allowable limit for axle groups by simple multiply the Rating Factor for the specific GAV vehicle by the maximum mass limit of each axle group.

- If RF for GAV Semitrailer <1.0; but RF for B-Double 50t >1.0
  - Single axle:

6t, unless otherwise advise by the assessor

  - Tandem axle:

To be calculated by the assessor through re-modelling of GAV Semitrailer vehicle with different Tandem axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tri-axle groups as discussed above.

- Tri axle:

To be calculated by the assessor through re-modelling of GAV Semitrailer vehicle with different Tri axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tandem axle groups as discussed above.

- If RF for GAV Semitrailer >1.0; but RF for B-Double 50t <1.0
  - Single axle:

6t, unless otherwise advise by the assessor

- Tandem axle:

To be calculated by the assessor through re-modelling of B-Double 50t vehicle with different Tandem axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tri-axle groups as discussed above.

- Tri axle:

To be calculated by the assessor through re-modelling of B-Double 50t vehicle with different Tri axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tandem axle groups as discussed above.

- If RF for GAV Semitrailer <1.0; and RF for B-Double 50t <1.0
  - Single axle:

6t, unless otherwise advise by the assessor

- Tandem axle:

To be calculated by the assessor through re-modelling of GAV Semitrailer and B-Double 50t vehicles with different Tandem axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tri-axle groups as discussed above.

The less value from the re-modelling of both GAV Semitrailer and B-Double 50t vehicles to be reported as the Tandem axle allowable limit.

- Tri axle:

To be calculated by the assessor through re-modelling of GAV Semitrailer and B-Double 50t vehicles with different Tri axle group loads to achieve a Rating factor of unity, in the presence of both Single and Tandem axle groups as discussed above.

The less value from the re-modelling of both GAV Semitrailer and B-Double 50t vehicles to be reported as the Tri axle allowable limit.

Chapter 9 Underbridge Loadings

C9-1 Changes in design loads

Over the years, design loads have changed as design codes have developed. Underbridge design loads have been expressed as:

- Cooper E (imperial)
- Metric Cooper M
- 300-A-12
- 300LA

Ratings for CRN underbridges are generally expressed as M or LA loadings.

Most bridges have been designed to older design codes and do not necessarily comply with the current design code.

Details of the changes in loadings are given below.

C9-1.1 ANZRC Rail Bridge Design Manual (1974)

The Australian and New Zealand Railway Conferences (ANZRC) Railway Bridge Design Manual Metric Cooper M loading is an approximate metrication of the American Railway Engineering Association (AREA), Iron and Steel Structures, Concrete Structures and Foundations, Cooper E loading, which was imperial. The maximum design live load in the state railway systems was AREA E60. This was approximately metricated to ANZRC M267 that was usually rounded off to M270. The ANZRC gave the recommended design load as M250 as shown in Figure 13.

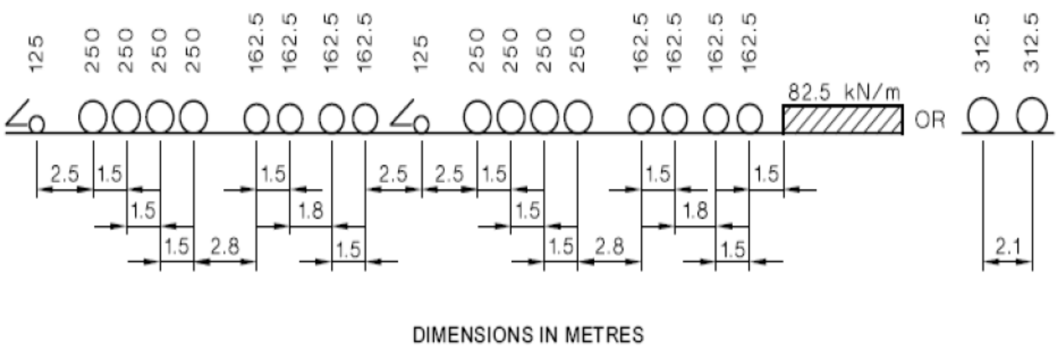


Figure 13 – M250 Live Load

C9-1.2 Australian Bridge Design Code (1996) – Railway Supplement

The 300-A-12 loading consists of groups of four axles each having a load of 300 kN, and having axle spacing of 1.7 m, 1.1 m and 1.7 m as shown in Figure 14.

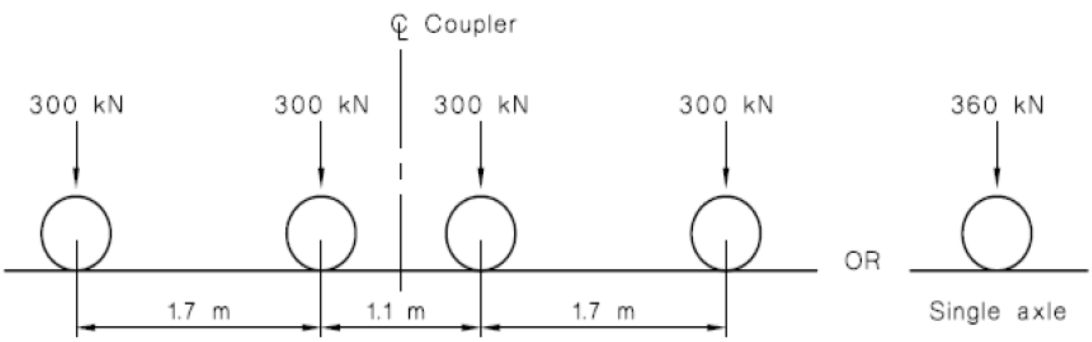


Figure 14 – 300-A-12-Axle Loads

The spacing between each Axle load should be taken as 12m (see Figure 15).

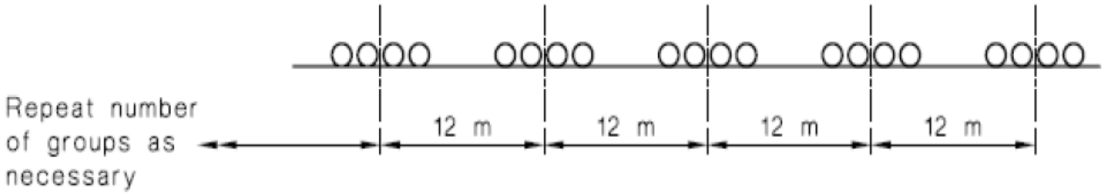


Figure 15 – 300-A-12-Axle Group Spacing

The 300-A-12 also includes a single axle load of 360 kN. The single axle load is not applied concurrently with other vertical railway live loading.

C9-1.3 AS 5100 Bridge Design Code (2004)

Figure 16 shows the 300LA loading which is the design load adopted from 2004 for bridge design.

This is a standard design loading (live load) and is meant to represent the worst-case loading and load configuration that a bridge will be subjected to.

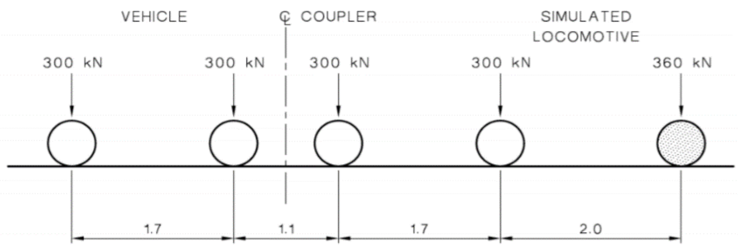


Figure 16 – 300LA Railway Traffic Loads – Axle Loads

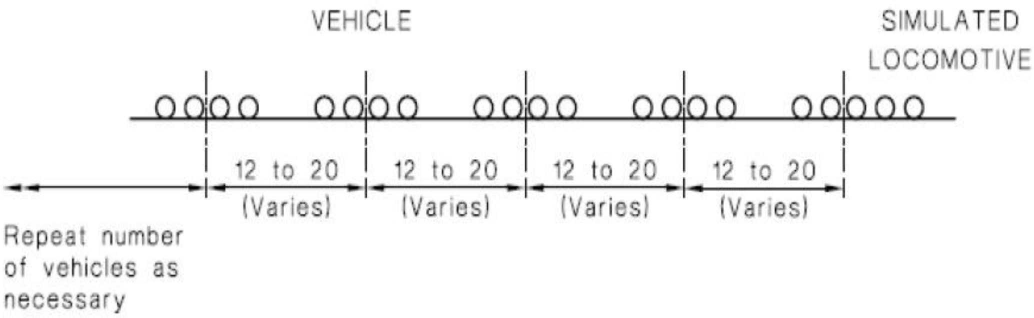


Figure 17 - 300LA Railway Traffic Loads – Axle Group Spacing

Load ratings of bridges are now to be related to the 300LA loading. Computation is performed for every critical structural element with the load capacity being determined as a proportion of the 300LA loading. The lowest load capacity of any element within the bridge is that quoted as the rating of the bridge, e.g. 225LA.

This methodology may simply be expressed as:

$$\text{LOAD RATING} = \{P/(1 + \alpha)\} \times 300\text{LA}$$

where P is the minimum of the proportions of static 300LA loading effect which can safely be carried by the structural elements in the bridge and α is the dynamic load allowance as set out in AS 5100.

Note that a load rating of 225LA therefore means 225L plus relevant dynamic load allowance.

Irrespective of the code or standard referred to, the higher the number the stronger the bridge i.e. it can carry higher loads and has more ability to withstand the effects of defects.

The design loads given below cover only the major vertical loads. They do not include dynamic load allowance (impact).

Note that dynamic load allowance generally increases with older codes as older non-dynamically balanced steam locomotives generated higher dynamic loads.

For underbridges, the minimum design loads are defined in CRN CS 310

C9-2 Loads and loading factors

Rating shall be undertaken using the loads and load factors in accordance with AS 5100 except as detailed below.

C9-2.1 Dead loads

The combined unfactored dead load of rails, guard rails and transoms of the track together with steel walkway(s) shall be taken as 5kN/metre.

C9-2.2 Live loads

The rating shall be derived from calculations based on the 300LA design loading in AS 5100, including 360kN front axle of simulated locomotive. The worst load effect shall be considered.

Ratings shall be specified in terms of current trains operating on the network. The following are recognised main line train consists on the CRN network and are shown diagrammatically as bellow.

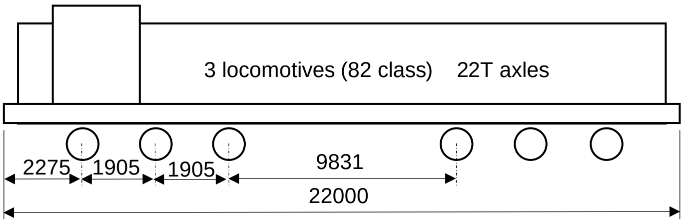
- Main Line freight (MF) - based on main line (82 class) locomotives plus 100 tonne NHGF coal wagons
- Branch line freight (BF) - branch line (422) locomotives plus 81 tonne NGTY wheat wagons
- Light Branch line freight (LB) - branch line (48) locomotives plus 76 tonne NGTY wheat wagons
- XPT/eXplorer (XP)
- Short bogie (SB92) - string of 11 metre bogie wagons such as RCGF steel coil wagons and a number of open wagons used in ore transport.

In addition to 300LA the following loads shall be used in rating.

- Class 1 & 2 lines – MF
- Class 3 lines - BF
- Class 5 – LB

C9-2.2.1 Loading diagrams

MF loading



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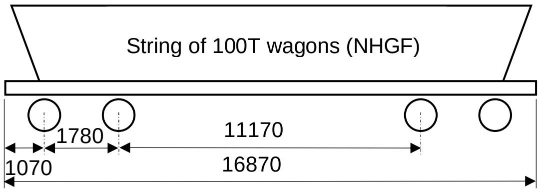
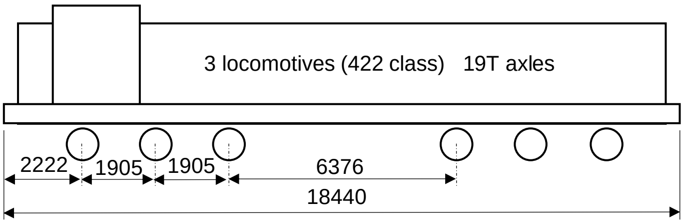


BF loading



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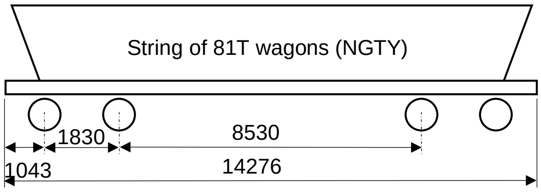
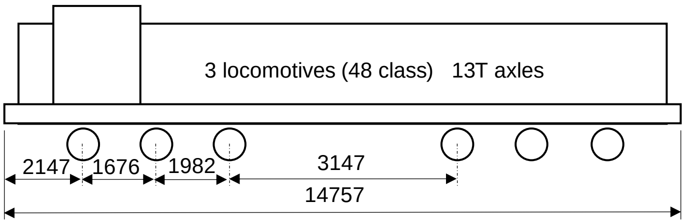


LB loading



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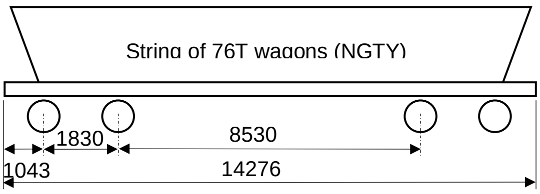
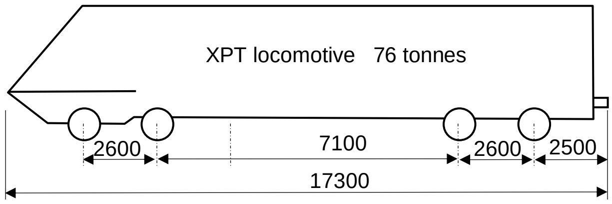


XP loading



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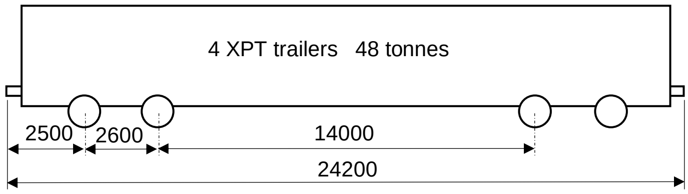
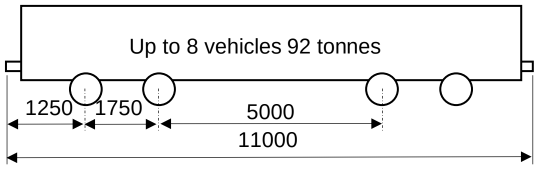


SB92 loading



C9-3 Comparison of 300 LA with M270

It is useful to compare the 300LA based ratings with previous M270 based ratings.

The comparison can be carried out using the Equivalent Base Length concept described in AS 5100 or by direct comparison of the load effects (e.g. bending moment and shear) of the 300LA and M270 live loads.

Note that care will need to be taken to allow for possible different impact values which may be associated with previous M270 based ratings compared with ratings based on AS 5100.

C9-4 Specific live loadings

If it is required to assess a bridge against a specific live loading, the procedure is the same as in the preceding sections.

Note that the Principal Track and Civil Engineer may approve reduction of load factors for specific live loads with the bridge capacity being, in effect, increased for the specific live load. Direct comparison of the load effects of the specific live load with the bridge rating for general traffic may therefore not be appropriate.

Where specific railway loads are used for the load rating work, an ultimate limit state load factor of 1.4 is permitted for the design case and 1.4rm when direct measurement is used, where rm is the ratio of the measured action to the action determined analytically. The value of rm may be less than unity.

C9-5 Choice of load factors

C9-5.1 General

Assigned load factors for load rating of existing bridges are based on the degree to which actual loadings are measured for a particular bridge. Dead and Superimposed Dead Load can be relatively easily and accurately estimated. Particular notice must be taken of the position and effects of services which have been added during the life of the bridge. In view of this, the Principal Track and Civil Engineer may approve reduction of dead load factors from the values used for design of new bridges.

General live loading is less predictable such as the possibility of overloaded wagons. Therefore, the rating live-load factor must be much the same as that for design, except in the case of a Specific Live Loading (see section C9-4).

C9-5.2 Load factors

Load factors for dead loads and railway traffic shall be in accordance with AS 5100.

Where the load carrying capacity rating of a component or connection is less than unity (1.0), the load factor for Live Load (LL) shall be calculated based on rating being equal to unity (1.0).

For example, if rating = 0.8 with LL load factor = 1.4, then LL load factor will be less than 1.4 for rating = 1.0.

The Principal Track and Civil Engineer shall determine if a load factor lower than the AS 5100 value of 1.4 is acceptable.

C9-6 Dynamic load allowance

The dynamic load allowance (DLA) specified in AS 5100 shall be used in the assessment of railway bridges.

For standard track, the dynamic load allowance is constant for speeds above 80km/hr, and varies linearly from zero for a speed of 0km/hr to the full value at 80km/hr. Therefore, in assessment of bridges for a speed greater than 80km/hr, the dynamic load allowance is the same as that for 80km/hr.

C9-7 Nosing load

For nosing load other than for 300LA traffic loads, the load shall be taken as the proportion of the heaviest axle load to the 30 tonne axle design load (e.g. for 100t wagons with 25t axle loads, the nosing load would be 25/30 x 100 = 83.3 kN).

C9-8 Wind load

A Serviceability Wind Speed of 20m/sec shall be used because of the short-term nature of the train loading on the structure.

Chapter 10 Rating steel and wrought iron bridges

C10-1 Rating requirements

For the superstructures of steel and wrought iron bridges, the load rating shall also be carried out in accordance with the requirements in this document.

Unless otherwise specified, all components and connections (including splices) shall be analysed.

C10-2 Steel bridges

The Limit States approach given in AS 5100 is to be adopted to load rate existing steel bridges where the following condition is required to be satisfied.

$$S^* \text{ (Design Action Effect)} < \phi R_u \text{ (Design Capacity)}$$

Where

S^* = Sum (load factors x nominal loads)

R_u = Nominal Capacity

ϕ = Capacity Reduction Factor

Load factors and the Capacity Reduction Factor φ are obtained from AS 5100.

The rating equation therefore becomes:

$$\gamma_g S^*_{DL} + \gamma_{gs} S_{SDL} + \gamma_L S^*_{LL+\alpha} = \phi R_u$$

Where

γ_g , γ_{gs} , γ_L = Load factors for dead load, superimposed dead load and live load respectively.

S^*_{DL} , S_{SDL} , $S^*_{LL+\alpha}$ = Nominal loads for dead load, superimposed dead load, and live load plus dynamic load allowance

Ru is dependent on material yield strength and geometry.

C10-3 Load capacity

In the absence of test data or designated steel type (on drawings or in specifications) the following values shall be used.

Material	Yield (MPa)	Ultimate (MPa)	Elongation (%)	Capacity factor, ϕ
Plates and sections				
Wrought iron (1)(2)	190 longitudinal 150 transverse	300	10	0.85
Steel<1910 (2)	210	400	20	0.90
1910-1940 (2)	230		20	
1941 – 1969 (2)	240		20	
After 1970	250		20	
Rivets (3)				
Wrought iron	Use same properties as for plate			0.8
Steel	Use same properties as for plate of relevant period			0.8

Table 8 – Material Factors

Notes **1.** Plastic properties not to be used if elongation <5%

2. Reduce yield by 5% where sections >20mm thickness are used

3. Field/hand driven rivets are assumed to be equivalent to shop rivets. All rivets, irrespective of installation method, have demonstrated satisfactory performance over the years.

Where testing to determine material tensile properties is undertaken, the requirements of AS 1391 “Metallic materials - Tensile testing at ambient temperature” shall be met. In the case of wrought iron, the additional requirements set out in Section C2-9.2 shall be satisfied.

C10-4 Loss of section

“As is” ratings shall be based on site measurements including losses of structural cross section due to corrosion or other causes.

The losses adopted in calculations shall be clearly stated and justified.

Where “as is” ratings are based on qualitative defect descriptions from inspection reports, use the losses detailed in Table 9.

Loss Level	Losses as a percentage of thickness
Minor	10%
Moderate	20%
Heavy	40%

Table 9 – Loss levels for “as is” ratings

An appropriate level of judgement shall be used in adopting a loss level. As an example, minor corrosion in the horizontal leg of an angle would imply a 10% loss in thickness of that leg.

C10-5 Wind and sway bracing

The wind and sway bracing on old steel structures consists of flat bars and angles which generally are found to not have adequate theoretical capacity for current rail traffic. However, there is no evidence that the bracing is being overloaded. Loading effects arising from dynamic load allowance are not applied to the bracing when calculating ratings.

The rating of these components will generally be less than one. The rating report shall include recommendations on the appropriate maintenance strategy i.e. inspection frequency, intervention levels and response times necessary to maintain safety.

C10-6 Wrought iron and cast iron bridges

The correct identification of the materials is critical to accurate rating calculations.

Provided that the testing of material properties and ductility checks have been carried out in accordance with AS 5100, the load rating methodology for wrought iron and cast iron bridges would be similar to that for steel bridges.

There is a much higher probability of material defects substantially affecting the strength of these members. The results of detailed inspection and non-destructive testing, where necessary including chemical analysis and micrographs, need to be considered in the assessment of these structures.

The appropriate Capacity Reduction Factor ϕ is obtained from AS 5100.

Refer also to Appendix 1 for general comment on these forms of construction including the possible need to reduce ϕ for certain poor quality wrought irons.

C10-7 Inadequate load capacity under existing conditions

Load-carrying capacity of existing steel Underbridges can be derived using AS ISO 13822 provided the original physical and structural integrity of the member under consideration has not been significantly altered and similar traffic conditions prevail.

Traffic Conditions for main lines

- Train configurations documented in Section C9-2.2 apply
- Performance shall be based on at least the past 20 years

Member Conditions

- The original physical characteristics and structural integrity of the member have not been altered by either strengthening or replacing it
- The member has not suffered more than 10% loss in capacity when load rated using dynamic load allowance factor (impact) from the ANZRC Railway Bridge Design Manual (1974)

Where the above traffic and member conditions for the application of AS ISO 13822 cannot be attained then the load-carrying capacity of that element shall be carried out using the dynamic load allowance from AS 5100.

The load-carrying capacity of Overbridge elements shall be carried out using the dynamic load allowance from AS 5100.

Chapter 11 Rating concrete bridges

The Limit States approach given in AS 5100 is to be adopted to load rate existing concrete bridges where the following condition is required to be satisfied.

$$S^* \text{ (Design Action Effect)} < \phi R_u \text{ (Design Capacity)}$$

Where

S^* = Sum (load factors x nominal loads)

R_u = Nominal Capacity

ϕ = Capacity Reduction Factor

Load factors and the Capacity Reduction Factor ϕ are obtained from AS 5100.

The rating equation therefore becomes:

$$\gamma_g S^*_{DL} + \gamma_{gs} S_{SDL} + \gamma_L S^*_{LL+\alpha} = \phi R_u$$

Where

γ_g , γ_{gs}, γ_L =Load factors for dead load, superimposed dead load and live load respectively.

S^*_{DL} , S_{SDL} , $S^*_{LL+\alpha}$ = Nominal loads for dead load, superimposed dead load, and live load plus dynamic load allowance.

R_u is dependent on properties of concrete, reinforcement and tendons, geometry, ultimate moment, and shear and torsional capacity.

The above methodology applies to reinforced, prestressed and partially prestressed concrete bridges.

Chapter 12 Rating timber bridges

C12-1 General

Timber bridges shall be rated using limit states methods in accordance with AS 5100 and AS 1720.1.

The rating methodology is limited to the load rating of existing standard timber bridges. It should be treated with caution due to variations in timber properties and bridge details.

C12-2 Standard timber underbridges

Standard timber underbridges provide a guide reference if onsite measurements are taken into consideration. They include the following.

C12-2.1 Superstructure types

Span (m)	Top	Designation
3.2	Transom	3.2TT
4.3	Transom	4.3TT
7.3	Transom	7.3TT
3.6	Ballast	3.6BT
4.6	Ballast	4.6BT
7.9	Ballast	7.9BT

Table 10 – Timber girder underbridge superstructure types

C12-2.2 Substructure types

Type	Top	Piles	Cross Brace
1	Transom	3	Single
2	Transom	5	Double
3	Ballast	4	Single
4	Ballast	6	Single

Table 11 – Timber girder underbridge substructure types

C12-3 Rating parameters

Timber bridges shall be analysed using parameters based on AS 5100 and AS 1720.1, as modified in the following sections.

C12-3.1 Dead loads

The minimum dead load per unit volume of any timber component shall be taken as 11 kN/m3.

The design dead loads and superimposed dead loads shall be obtained by applying the appropriate load factor to the nominal loads on the structure. Applicable load factors are in Table 3 for underbridges and Table 5 for overbridges.

Where the dead load is calculated from the dimensions shown on the drawings, the “design case” load factor applies. Where an assessment of an existing member is being undertaken, and dead load is calculated from actual dimensions measured on site, the “direct measurement” load factor applies.

C12-3.2 Live loads

The design vehicles to be used in the analysis shall be as advised by the Principal Track and Civil Engineer, with Ultimate Limit State Factors and Dynamic Load Allowances as per the relevant Standards.

The ultimate design live load action is equal to:

(1 + DLA) x load factor x action under consideration.

Where the applicable load factors are in Table 3 for underbridges and Table 5 for overbridges.

C12-3.2.1 Underbridge Live Loads

Railway load configurations used for assessment shall be as per Chapter 4 and Chapter 9, unless otherwise advised by the Principal Track and Civil Engineer, together with the load factors in Table 12 below.

Type of Load	Ultimate Limit States	
	Where Load Reduces Safety	Where Load Increases Safety
Dead load (design case)	1.4	0.8
Dead load (direct measurement)	1.2	0.9
Superimposed dead load (general loads)	2.0	0.7
Superimposed dead load (controlled case)	1.4	0.8
Railway loading (general loads)	1.6	N/A
Railway loading (specific loads)	1.4	N/A
Centrifugal and nosing forces	1.6	N/A
Braking and traction forces	1.6	N/A

Table 12 – Ultimate limit State Load factors for underbridges

C12-3.2.2 Overbridge Live Loads

Roadway load configurations used for assessment shall be as per Chapter 4 and Chapter 8, unless otherwise advised by the Principal Track and Civil Engineer, together with the load factors in Table 13 below.

Type of Load	Ultimate Limit States	
	Where Load Reduces Safety	Where Load Increases Safety
Dead load (design case)	1.4	0.8
Dead load (direct measurement)	1.2	0.9
Superimposed dead load (general loads)	2.0	0.7
Superimposed dead load (controlled case)	1.4	0.8
Roadway loading (general loads)	2.0	N/A
Roadway loading (specific loads)	1.5	N/A
Centrifugal and nosing forces	Refer to AS 5100.2	N/A
Braking and traction forces	Refer to AS 5100.2	N/A

Table 13 – Ultimate limit State Load factors for overbridges

C12-3.3 Capacity factors (ϕ)

Values of capacity factor (ϕ) for calculating the design capacity of structural members (R_d) and structural joints (N_d) shall be taken from AS 5100 and AS 1720.1.

For example:

ϕ = 0.75 for sawn timber

ϕ = 0.60 for round timbers

ϕ = 0.60 for bolts larger than M16

ϕ = 0.75 for bolts M16 and smaller

Values of capacity factor (ϕ) for calculating the design capacity of secondary members (such as deck planking, sheeting, timber railings, or other members whose failure could not result in collapse of a significant portion of the structure) or joints in such members may be taken from AS 5100 and AS 1720.1 (secondary members in structures other than houses).

C12-3.4 Characteristic values for load rating

The characteristic strength properties in bending, tension, compression and shear and characteristic stiffnesses for the design of structural timber elements shall be taken from AS 5100 and AS 1720.1.

In the absence of data, the timber shall be assumed to be Stress Grade F22, Strength Group S1.

The relevant portion of AS 5100 and AS 1720.1 is replicated in Table 14 - Characteristic Values for F22 Stress Grade Timber (MPa)below, with notes as follows:

- The characteristic values in Table 14 for bending apply to beams not greater than 300 mm in depth. For beams greater than 300 mm depth, the characteristic values shall be obtained by multiplying the value in Table 14 by (300/d)0.167, where ‘d’ is the depth of the section
- The characteristic values in Table 14 for tension, apply to tension members with largest cross-sectional dimension not greater than 150 mm. For tension members with a cross-sectional dimension greater than 150 mm, the characteristic values shall be obtained by multiplying the value in Table 14 by (150/d)0.167, where ‘d’ is the width or largest dimension of the cross-section

Stress Grade	Bending (f'_b)	Tension Parallel to Grain (f'_t)	Shear in Beam (f'_s)	Compression Parallel to Grain (f'_c)	Modulus of Elasticity Parallel to Grain (E)	Modulus of Rigidity (G)
F22	55	34	4.2	42	16,000	1,070

Table 14 - Characteristic Values for F22 Stress Grade Timber (MPa)

C12-3.5 Duration of load factor k_1

Values for the duration of load factor k_1 for the strength of timber shall be as follows:

k_1 = 0.57 for permanent actions e.g. dead load, superimposed load, loads due to earth pressure

k_1 = 0.97 for ultimate live load

Values for k_1 for the strength of joints with laterally-loaded fasteners shall be as follows:

k_1 = 0.57 for permanent actions e.g. dead load, superimposed load, loads due to earth pressure

k_1 = 0.86 for ultimate live load

Note that in accordance with AS 5100 and AS 1720.1, for any given combination of loads of differing duration, the factor k_1 to be used is that appropriate to the action that is of the shortest duration. For example, when considering ultimate dead load plus ultimate live load, the appropriate member k_1 factor is 0.97.

Generally, the forces due to dead load in most timber elements in a bridge are quite small compared to those caused by live loads. However, some components in large span trusses may be subjected to relatively high dead-load forces. Dead load should, therefore, also be considered by itself or combined with other permanent loads in such cases using k_1 of 0.57 for permanent actions.

C12-3.6 Temperature factor k_6

For the assessment of timber bridges in New South Wales, the temperature factor (k_6) shall be taken a s 1.0.

C12-3.7 Strength sharing factor k_9

For the assessment of timber bridges, the strength-sharing factor (k9) shall be taken as 1.0.

C12-3.8 Modification factors k_4, and k_{12}

Modification factors k_4 (partial seasoning factor) and k_{12} (stability factor) shall be in accordance with AS 5100 and AS 1720.1.

C12-3.9 Round timbers

Where round timbers are used (such as in pier trestles or girders), these shall be assessed in accordance with AS 5100 and AS 1720.1. Where these members are shaved on one or more faces, assume that the shaving will reduce the modulus of elasticity by 5% in accordance with AS 5100 and AS 1720.1. The shaving factor k_{21} shall be taken from Table 6.3, except for the case of bending where only the compression face of the round timber is shaved. For this case, k_{21} may be taken as 0.95. This situation will commonly occur in the case of girder spans, where the tops of the girders are shaved to provide a flat bearing surface for the transoms or decking.

C12-3.10 Transverse load distribution

Determine live load distribution to load carrying elements by detailed analysis (e.g. grillage analysis).

The grillage model should include transoms or decking and cross girders where appropriate in the overall load carrying system, but not the rails.

C12-3.11 Girder composite action

Assume that double girders do not act compositely even in the case of bridges where timber block shear keys have been incorporated. It is considered that timber dimensional changes, local crushing and bolt loosening would render this system unreliable.

C12-3.12 Continuity corbel effect

The flexure continuity effect of the corbels shall be accounted for by using the following factors on the simply supported span bending moments (see Table 15).

Span (m)	Girder continuity type		
	Single	End	Intermediate
3.6 - 4.6	0.90	0.80	0.75

7.3 – 7.9	0.95	0.90	0.85

Table 15 - Continuity Corbel Effect

Note: ‘Single’ denotes a single span timber overbridge

‘End’ denotes an end span of a multiple span timber overbridge

‘Intermediate’ denotes an inner span/s of a multiple span timber overbridge.

C12-3.13 Corbel bending

Bending in the corbel shall be calculated assuming the girder reaction is applied at a distance of 0.6m from the effective support.

C12-3.14 Centrifugal force factors

The centrifugal force factors shall be as per AS 5100.2.

C12-3.15 Soil pressure

The soil pressure at a depth “h” behind sheeted abutments shall be based on the following formula:

$$P = K_a \chi h \text{ with } K_a = 0.5$$

Where

χ = soil density = 20 kN/m^3 and “h” is the abutment height.

C12-3.16 Live load surcharge pressure

The live load surcharge pressure behind sheeted abutments at increasing depth in fill due to vehicle loading shall be computed in accordance with AS 5100.2.

For horizontal load, multiply by the appropriate earth pressure coefficient.

C12-3.17 Calculation for ‘as is’ conditions

Defects, including pipes and surface troughs, shall be accounted for in the ‘As Is’ ratings by reducing the section properties of the ‘As New’ members accordingly. A precise analysis shall be undertaken for accurate calculation of the pipes effect on section capacity.

C12-4 Non-standard rating parameters

All standard timber bridges shall be load rated in accordance with the above. For non-standard structures, or for standard structures where aspects of the rating cannot be complied with or are not adequately covered, the Principal Track and Civil Engineer will provide advice.

Chapter 13 Rating masonry arch bridges

A similar Limit States methodology to that described for steel and concrete bridges shall be adopted to determine the load capacity of masonry arches. Additional load effects due to earth pressure and high, superimposed dead loads should be taken into account and higher load factors should be adopted for dead loads to reflect the greater degree of uncertainty associated with the determination of these loads than for the steel bridges.

Frequently the existing rail level is higher than the design rail level. This may affect the strength and stability of the balustrades and spandrel walls, and if so, should be reported with the rating.

The rating equation can be given as:

$$\gamma_g S^*_{DL} + \gamma_{gs} S^*_{SDL'} + \gamma_e S^*_{EP} + \gamma_L S^*_{LL+\alpha} = \phi R_u$$

Where

γ_g , γ_{gs}, γ_L = Load factors for dead load, superimposed dead load and live load respectively in accordance with AS 5100.

γ_e = Load factor for earth pressure in accordance with AS 5100

S^*_{DL} , S^*_{SDL} , $S^*_{LL+\alpha}$ = Nominal loads for dead load, superimposed dead load, and live load plus dynamic load allowance.

S_{EP} = Nominal load for earth pressure and could be a maximum or minimum load effect with appropriate load factor, γ_e

ϕ = Capacity reduction factor obtained from Table 4.1 of AS 3700.

R_u = Nominal Capacity dependant on characteristic compressive strength (f'm) of the brickwork (10 MPa recommended from tests) and geometry.

The arches can be analysed using a structural analysis program such as "Microstran" or “SPACE GASS”, with the following assumptions:

- Structural action of spandrel walls and balustrades ignored
- Arching action of any concrete-based fill ignored
- Arch section is uniform
- Arch is fixed in direction at the springing points

Limiting Stresses

The stresses of the intrados and extrados of the arch are calculated based on the following:

(axial force / area) ± (bending moment / modulus of section)

Where the section is entirely in compression and within the ultimate limiting value the section is considered to be satisfactory.

Where one extreme fibre of the section is in tension and the other side is in compression, it is assumed that the section does not have any tensile capacity and is cracked. The compressive stress is then recalculated based on a cracked section see Figure 18.

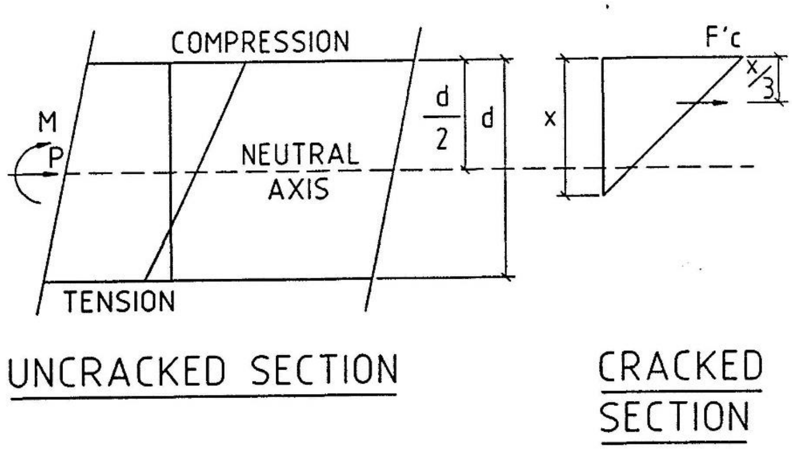


Figure 18 - Masonry Stress Diagrams

'b' = width of arch barrel (in consistent units).

Applied axial force	(P) = 1/2 F'_c.x.b
Applied moment	(M) = P[(d/2) –(x/3)]
Rearranging:	X = 3[(d/2) –(M/P)]
And	F'_c = 2P/(xb)

If the compressive stress F'_c is within the ultimate limiting value, the section is considered to be satisfactory.

Chapter 14 Field testing

C14-1 Load testing

Determination of the live load capacity of a bridge can also be assessed by the test loading of the bridge. Test loading would generally be considered where:

1. The bridge rating cannot reliably be determined analytically. For example, there may be some doubt about member properties, or the bridges observed capacity may be significantly different from its theoretical rating.
2. The theoretical rating is low and bridge renewal is likely to be expensive or disruptive.

Past experience with load testing has been that bridges typically have a greater capacity than that predicted by theory.

The following general methodology applies to assessing the determination of bridge capacity by load testing.

1. Inspection to determine bridge condition including identification of section loss and also to confirm details shown on the drawings.
2. Pre-analysis of the bridge to determine theoretical ultimate, proof and rating loads and theoretical modes of failure.
3. Static test loading up to or above the theoretical proof load using instrumentation to measure and "real time" monitor strains and deflections and to compare with theoretical values.
4. Post analysis of the bridge taking into account data obtained from the load test.
5. Determination of the bridge rating based on load testing.

C14-2 Strain gauging

C14-2.1 General

Strain gauging is a very valuable tool to assist with load and fatigue rating.

C14-2.2 Strain gauge recording

It is essential to have a continuous graphical recording of the strain gauging at an appropriate speed to show all short duration, dynamic loads from wheel defects at high speed. Whenever possible a magnetic trace of the strain gauging should be made. This will assist in reprinting graphs at various speeds, where required.

C14-2.3 Preparation for strain gauging

Prior to determining locations for strain gauges, check the current bridge examination report. Collect all relevant historical information on such things as loose rivets, history of cracking and previous repairs and strengthening.

Inspect the structure for signs of high dynamic load and fatigue problems. Look for cracks, loose rivets and broken bolts, particularly at bridge ends, also in members and bracing close to the track where dynamic load is highest. Check for any flat bar primary or bracing member, or any member not complying with design requirements for stiffness, which may resonate under dynamic load.

C14-2.4 Strain gauge locations

From history and inspection information, select strain gauge locations for maximum stress, and maximum dynamic load.

- Strain gauge the end connections of cross girders, stringers and similar members for moment
- Strain gauge bridge ends, areas near wheel loads and areas where cracks, loose rivets or broken bolts have occurred

- For through-girder overbridges, the stringers and cross girders adjacent to abutments will have maximum dynamic load and connections on these members should be strain gauged for moment
- For deck girders or trusses the top flange or top chord will have high dynamic load, at the end of the span where vehicles approach, at the point of high torsional load under transoms. Similarly, the end-sway brace may also have dynamic load. All flat-bar members can be expected to resonate and record strains well above the predicted dynamic load

Ensure sufficient strain gauges are selected to check the accuracy of the analysis model.

C14-2.5 Loading for strain gauging

Apply loads incrementally from 50% of theoretical ultimate capacity. Monitor responses to ensure that the bridge is responding in a linear-elastic manner.

A captive train (made available for the full testing regime) equal to or close to the maximum loading to be used on the overbridge is best for strain gauging. Record the captive train at crawl speed, 10 km/hr and at 10 or 20 km/hr intervals to line speed (or higher under special circumstances). Also ensure that sufficient general traffic at line speed is recorded including both disc-braked and tread-braked vehicles with worn wheels. If possible, record 20 general traffic trains to assess the proportion of worn wheels and train types causing high dynamic loads.

Where use of a captive train is not practical, general traffic will have to be recorded, as above. It is highly desirable to arrange to run some trains at 10, 20 and 40 km/hr as well as line speeds.

Note that in order to confirm the analysis model some reasonably accurate estimate of actual axle loads will be required.

C14-2.6 Comparison of computed stress histories and strain gauging

Compare computed stress histories and strain gauging preferably at crawl speed or at 10 km/hr. If there is a good correlation, then the analysis model is proved. If not, the model may require adjustment.

C14-2.7 Determination of dynamic load from strain gauging

For a captive train compare the crawl (or 10 km/hr) strain gauging with that at the speed at which dynamic load is required to be assessed. The increase in strain represents the dynamic load effect for that speed.

Where a captive train was not used and general traffic was strain gauged; at the point on the graph where the maximum strain is recorded and the mean strain is the maximum, the dynamic load should be determined. The mean, between maximum and minimum pulse, should be compared to the maximum to determine the dynamic load at speed.

Determine the percentage of trains with defective wheels and high dynamic load, and record the train type, for use in fatigue analysis. Where defective wheels cause more than one pulse cycle (where analysis indicates one cycle), allow for the additional cycles in the fatigue analysis.

In cases where resonance occurs in members, determine the number of cycles that occur where analysis determines one cycle and allow for the additional cycles in the fatigue analysis.

Tabulate dynamic load versus speed for relevant members and connections. This is particularly useful for determining speed limits for marginal structures.

Appendix 1 Commentary on steel and wrought iron structures

A1-1 Cast Iron

Cast iron girders are of particular concern due to their lack of ductility at all temperatures. For tests performed on existing and removed girders, all have an unacceptably high phosphorous content which maximises brittleness. The phosphorous contents are all well above the maximum permitted in current Australian Standards. Sand inclusions and other defects have been found in all previously tested girders, further increasing the probability of brittle fracture.

Cast iron substructures have not been examined to the same extent, but their brittleness is not considered to be a problem as long as they remain stressed in compression, or with minimal tension and no impact loading is applied.

Graphitisation is the main concern with cast iron substructures. It occurs at or below water level when the iron is corroded out leaving a matrix of graphite, which appears unchanged from the original cast iron. Site inspection of graphitisation should be done by tapping the cast iron with a geology pick. Assessment of cast iron's susceptibility to graphitisation can be done by metallurgical examination of micrographs.

A1-2 Wrought Iron

Wrought iron is often mistaken for modern 250 grade steel, with serious over-rating resulting. Almost all NSW rail bridges constructed up to 1891 were constructed of wrought iron, including lattice girders and major trusses. Plate-web girders in wrought iron were constructed up to at least 1894. It is not acceptable to assume the drawing dates after 1894 indicate steel, as some drawings were prepared from measuring the existing structure many years later and dated with the date of measurement.

Note that some wrought iron bridges have had stringers, cross girders and/or bracing replaced by steel, so identification must include inspection of the larger members.

Once identified as wrought iron, check the examination report or structure inspection for typical defects which will reduce the rating or fatigue endurance. Check for laminations perpendicular to the surface of rolling and that which intersects with rivet holes. Some laminations parallel to the surface of rolling can be several metres long. When rivets are found to be loose and replaced, check the rivet hole for laminations opening up as de-laminations, which may be precursors of fatigue cracks. Magnetic particle inspection and ultrasonic inspection will assist here. Wrought iron rivets which are cracking or "splitting" radially are likely to become loose at a later date, which is considered to indicate significant fatigue damage.

Wrought iron is very variable in its properties, having a much higher standard deviation on yield strength and ultimate tensile strength than steel. Similarly, ductility is very variable.

Elongation and nick bend tests should be used to evaluate the brittleness of wrought iron in bridges to be load rated or fatigue rated. The nick bend test is to be performed with a sample 30 mm wide, the original material thickness and 200 mm long. The nick may be a shallow saw cut as for a weld nick bend test. Where elongation is less than 10% and/or the nick bend test has more than 10% crystalline fracture, the possibility of brittle fracture shall be reported on. Where elongation is less than 5% and/or the nick bend test has more than 20% crystalline fracture, a special inspection of all of the structures areas where brittle fracture is possible, is to be performed with magnetic particle and ultrasonics testing. The structure is to be strain gauged to prove the analysis model, particularly with respect to continuity of joints, and acoustic emission testing is to be considered if the risk to life is considered significant. Consideration should be given to lowering the rating by using a capacity reduction factor of say 0.5 in this case. Where elongation is 1% or less, or the nick bend test has more than a 50% crystalline fracture, consideration of rating using a capacity-reduction factor of 0.33 should be made.

Acoustic emission is far more accurate for determining transition temperature in wrought iron than impact tests, as the impact test fracture surface will cross various slag layers in the wrought iron, but brittle fracture in the structure will run along slag layers, or de-laminations.

Where samples are to be tested for yield and Ultimate Tensile Strength (UTS), 10 samples are recommended as a minimum, with at least two from each angle thickness and flange-plate thickness represented in the structure. The mean minus two standard deviation value is recommended for rating. Two standard deviations are recommended as OneSteel in new steel production, in 10 and 12 mm plate achieves four standard deviations above the specified yield.

Wrought iron rivets must be rated as wrought iron and not as steel. Even this assumption may not be conservative considering the observed frequency of poor-quality wrought iron rivets.

Welding is not recommended for any wrought iron, as laminations in the heat-affected zone are likely to open up as de-laminations. Where the fusion zone is parallel to laminations, they are likely to open up allowing complete delamination from the weld.

A1-3 Brittle fracture

A1-3.1 General

Assess the possibility of brittle fracture for all cast iron, wrought iron and steel superstructures at the time of load rating and fatigue rating.

This is extremely important as brittle fracture travels through a structure at any temperature below its transition temperature, within milliseconds. This inspection cannot be used to detect the start of brittle fracture before it propagates to complete failure, as is the case with relatively slowly propagating fatigue cracks. Brittle fracture is one of the most likely causes of bridge collapse.

A1-3.2 Structures susceptible to brittle fracture

The following groups of structures have particular susceptibility to brittle fracture.

1. **Cast Iron Girders**

   Cast iron girders in existing overbridges are of such concern in relation to the possibility of brittle fracture that they have been continuously supported.

2. **Broad Flange Beams**

   BFBs have very variable notch ductility. They have the worst impact properties of any steel used in NSW railway bridges. Typical Charpy V-notch results are 4 to 5J at ambient temperature.

   Some Broad Flange Beams (BFB) have been subjected to high-impact road vehicle collision loads and have shown substantial plastic deformation. The manner in which they perform cannot be determined unless Charpy (or other impact tests) are done for each girder, preferably at 0°C and room temperature or additional temperatures to assess the transmission temperature. The transition temperature should be below the minimum service temperature. If not, the girder is to be considered brittle. In the absence of this test, all BFBs must be considered to be brittle.

   BFBs with welded cover plates require careful inspection of the transverse and tapered welds, with the aid of magnetic particle testing. If cracks are found, they should be further defined by ultrasonic testing, to assist in determining whether renewal or strengthening is required.

   Where BFBs are over roadways, and subject to vehicle impact, it is usual to recommend renewal. This applies to those with welded cover plates, welded repair or strengthening in vulnerable locations, defects such as cracks, rolling defects or impact damage in important locations and/or those which are brittle should be given the highest priority for renewal. If not renewed, crash beams to protect the girder are highly desirable.

Where BFBs are found to be very brittle, consideration should be given to lowering the rating by using a lower capacity reduction factor of say 0.5 or even lower, approaching that for cast iron.

3. Wrought Iron

The lack of ductility in some wrought irons is discussed above. All significant members in wrought iron rail bridges in NSW are of riveted construction. Thus, if a brittle fracture occurs, it will only propagate to the edge of that riveted component, and the maximum loss of flange area will be 50%. Total fracture leading to collapse should be delayed for some time, depending on loading. It is anticipated that inspection will find the fracture prior to total collapse.

Unfortunately, some wrought iron bridges have been repaired or strengthened by welding, and in some cases much worse situations possibly exist. Firstly, the weld probably will open some laminations as de-laminations which could propagate as brittle fractures. Secondly the welding may permit a brittle fracture to travel from one component to the next until complete collapse occurs. These aspects need to be considered in the rating and are to be reported.

4. Welded steel girders prior to 1966

Welded steel girders from 1966 onwards in overbridges were specified from steel designated as NDI or LO or L15 or tested to the standard for LO. Steel prior to 1966 should be assumed not to comply with these notch ductility requirements. When fabricated into girders by welding, the girders may have a significant probability of brittle fracture. Check the examination report for defects which may act as brittle fracture initiators. Report on the probability of brittle fracture.

5. Steel

Any as rolled, riveted or bolted member fabricated before 1966, that has been repaired or strengthened by welding, is likely to have an increased risk of brittle fracture. This is particularly true for riveted or bolted members where the welding will permit a crack to propagate beyond the edge of the original element, through the whole flange, or member.

Steel in bridges prior to 1940 can be considered not able to be welded, unless proved otherwise by weldability tests. Steel produced up to 1925 can be considered to be even less able to be welded. It should be noted that girders in jack arches will be in this category. Welds on these not able to be welded steels or wrought iron can be expected to have numerous heat-affected zone (HAZ) cracks. Some may be reported in the examination report. Others may not be detected unless magnetic particle or ultrasonic testing is performed. Where these HAZ cracks are perpendicular to significant tensile stresses, brittle fractures may occur.

A1-3.3 Types of dynamic loading for brittle fracture

The types of dynamic loading giving sufficiently rapid rates of strain to cause brittle fracture are as follows.

1. Road vehicle impact on overbridges and underbridges

Road vehicle impact with girders over roadways by high vehicles is the most common loading causing brittle fracture in NSW rail bridges. If the girder does not fracture in a brittle manner on the first impact but deforms with up to 10% outer bend fibre strain, the transition temperature will be raised by 20°C in the deformed area. If another high vehicle hits the deformed area before it has been repaired, the possibility of brittle fracture is much increased.

Repair of impact damage must be done by heating to above 500°C, straightening and grinding any notches. This should restore the original transition temperature. If the examination report or site inspection indicates this has not been done, then report accordingly.

2. Railway loading on overbridges and underbridges

Dynamic loading from defective wheels, wheel burns, temporary rail joints or broken rails are all able to cause brittle fractures in susceptible overbridges. Of these, a wheel burn on an overbridge is probably the most likely loading to cause brittle fractures. As wheel burns are most likely when a train starts after stopping at a signal, the proximity of the overbridge being rated to signals should be noted and reported if significant.

Ballast top overbridges will dissipate dynamic load in the ballast and decking. This will occur much more than in a transom top overbridge. Where transom top overbridges are a significant brittle fracture risk, fitting resilient support to transoms will reduce and dampen the dynamic load. Reducing train speeds is the simplest method of reducing dynamic load.

Collision damage from derailments are rare but should be treated similarly to road vehicle collision damage.

A1-3.4 Types of notches

The following types of notches, able to initiate brittle fracture, can be found in bridges. Those located in a plane across an area of significant tensile force are of most concern.

1. Poor geometric details

Poor geometric details may occur because of design or fabrication. Some example of such are coping at the end of stringers or girders cut square with no radius, rough oxy cut surfaces, and transverse welds with undercut at the end of a partial length cover plate.

2. Cracks

Cracks may be of the following types:

- Cracks in welding, most commonly in the heat-affected zone, but also in hot cracking
- Fatigue cracks
- Ductile-tearing cracks. These usually result from overload but may be from road-vehicle impact or train-derailment impact
- Rolling defect, lamination or casting defect, from the manufacture of steel, wrought iron or cast iron

3. Impact damage forming a notch

A notch is formed in many cases where plastic deformation occurs after impact with the bridge from a road vehicle, derailment, or part of a train or its load, becoming loose.

A1-3.5 Bracing systems

General

The bracing systems for both sway bracing and wind bracing are the most likely members to have the lowest rating on an underbridge.

Underbridges on curves

Underbridges on curves frequently have the wind bracing members oriented to be in tension with centrifugal force applied. For the bracing, the most critical loading is usually the design train at low speed with maximum nosing load acting towards the centre of the curve, resulting in compression in the wind bracing members.

For average radius curves, nosing load at about that specified in AS 5100, can act towards the centre of the curve at speeds down to about 10 km/hr. At speeds below 10 km/hr the nosing load drops off. It is recommended that this case be checked at 10 km/hr.

Flat bar bracing

Induced, very high-frequency dynamic loading in flat bar bracing members causes premature fatigue damage as well as frequent extensive plastic deformation. If they are not replaced a suitable system must be designed to re-tension them. Without such a tensioning system, the rating must be reduced considering the lateral girder movement that must occur before the bracing is stressed.

Bracing tensioned by turnbuckles

Bracing members tensioned by turnbuckles are a major maintenance problem, even when the strength of the bracing appears to be adequate. Once wear occurs at the pinned ends or the turnbuckle vibrates loose, it is usually very difficult to re-tension due to corrosion and build-up of paint in the turnbuckle thread. In some cases, vibration of loose bracing is so bad that nuts fall off pins and pins fall out. To re-tension turnbuckles it is usually necessary to disassemble and run a tap and die down both threads. The cost of this work is such that it is usually more economical to replace the bracing.

Where bracing remains loose, violent, lateral oscillation occurs with trains at speed in susceptible underbridges. Ratings should consider the effect of loose bracing. Where necessary, speed limitations should be made.

Welded bracing

Welded wind and sway bracing and diaphragms generally fail due to fatigue cracking earlier than the equivalent member if bolted or riveted. In addition, occasional loads above the load the bracing was designed for, may occur. This will result, at best, in plastic deformation of the bracing, but may cause ductile tearing cracks or even brittle fracture. Any crack may then propagate in fatigue. A bolted or riveted connection will usually slip or plastically deform resulting in loose fasteners, rather than cracking, when overloaded.

In most cases, bracing that was designed for welding when the bridge was new will perform much better than riveted bracing that has been repaired or strengthened by welding. This results in welds with a high incidence of HAZ cracking, and micro-cracking at the weld fusion zone. Much more rapid fatigue cracking will result, or possibly brittle fractures may occur in susceptible metals.

Fatigue cycles accumulate in bracing at least at one cycle per axle, but in some dynamic cases, at much higher frequency than this.

Ratings of underbridges and particularly fatigue ratings, must carefully consider welded bracing.

Appendix 2 Presentation of rating results

Executive summary

	Bridge Superstructure Member Rating			
Vehicle	Main Long. Girder	Primary X Girder	Secondary X Girder	Secondary Long. Stringer
T44 / 300LA	3.13	2.23	6.14	4.01

Introduction

Include here introductory paragraphs to the Report including a statement of the scope of work, locations and configurations of bridges that have been rated, general observations and comments etc.

Methodology and assumptions

Include here a statement regarding the methodology and assumptions used in the rating, including:

- General statement regarding methodology used in the rating
- Reference Standards used (e.g. AS 5100.7:2004/Amdt1 - 2010; AS 1170:2002; AS 4100:1998 etc.)
- Material factors adopted (e.g. yield stresses etc.)
- Loads and loading factors used.

Engineering details

Superstructure Connection Rating (Speed > 80km/hr)					
Primary X Girder To Main Box Girder (Bolts)	Long. Stringer to Primary X Girder (Rivets)	Secondary X Girder to Main Box Girder			
		Complete Connection		One Failed Web Cleat	
		Rivets	Cleats	Rivets	Cleats
5.11	1.08	1.06	1.11	1.04	1.01

Appendices

- Bridge photographs (along tracks & elevation)
- Bridge capacities
- Load effect summaries
- Inspection summaries
- Theoretical fatigue damage
- General Arrangement drawings