UGL REGIONAL LINX

UNDERBRIDGES – LOAD RATING

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CRN CM 306

LINKING COMMUNITIES. CONNECTING CUSTOMERS.

Table of Contents

Document Control

Function	Position	Name	Date
Approver	A&E Manager	Lucio Favotto	30.01.2022

Revision	Issue Date	Revision Description
1.0	30.01.2022	UGLRL Operational Standards Template applied
2.0	30.01.2022	First approved and issued UGLRL version for issue to website

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 rating or strength assessment of underbridges on the Country Regional Network (CRN).

Rating describes the method which derives the safe capacity of an underbridge to carry repeated live loading for its design life, according to code requirements.

The manual covers underbridges in a variety of materials and structural forms.

Materials covered include:

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

The aim is to supplement AS 5100 "Bridge design" in providing consistent and comprehensive rules and procedures for the rating of underbridges in both "as new" and "as is" conditions for materials other than timber, for which there is content in AS 5100. The procedures described rating of timber underbridges is based on AS 1720 "Timber Structures"

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.

C1-3 Who should use this manual

Load rating of structures on the CRN may only be undertaken by persons who have been granted appropriate Engineering Authority by the Principal Track and Civil Engineer.

This Manual should only be used by professional engineers, experienced in railway bridge design and assessment who have been granted engineering authority. Limit State Design "safety factors" are not intended to protect against errors arising from work not carried out with a reasonable standard of professional competence.

C1-4 Background

The Australian Bridge Design Code AS 5100 includes the design and rating of railway 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.

C1-5 How to read this manual

When you read the information in this manual, you will not need to refer to CRN Engineering standards. Any requirements from standards have been included in the sections of the manual and shown like this:

The following design requirements are extracted from CRN Engineering Standard CRN CS 310 "Underbridges"

Track Class	Design Load configuration
Main Lines	
Heavy Haul Coal Operations	350-LA plus DLA

Reference is, however, made to other manuals.

Throughout this manual reference is made to the following levels of Engineering Authority:

- Principal Track and Civil Engineer

C1-6 References

C1-6.1 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

C1-6.2 CRN documents

CRN CS 100 – Civil Technical Maintenance Plan

CRN CS 300 - Structures System

CRN CS 310 - Underbridges

CRN CM 001 – Civil Technical Competencies and Engineering Authority

CRN CM 302 – Structures Examination

CRN CM 305 – Structures Assessment

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

C1-6.3 Other references

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

American Railway Engineering Association (AREA) Railway Engineering Manual

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

C1-7 When is rating required?

All existing underbridges 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.

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”.

Load rating of timber underbridges shall be carried out with reference to AS 1720 Timber 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 and 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 ballast 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 one unit.

C2-6 Rating procedure

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

C2-7 Statement of load rating

Determination of bridge live load capacity is generally based on assessment of superstructure capacity. The superstructures of some bridges, particularly masonry arch bridges, are 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 "300LA plus"(i.e. undetermined but greater than 300LA).

Note that any relevant parameters such as ballast 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 railway loading, based on a load factor on live load (γL) of 1.6 for 300LA loading and 1.4 for specific (actual known) railway loading configurations.

However, a number less than 1 means either of the following scenarios:

- The bridge element under consideration does not theoretically satisfy the nominated railway loading, based on a load factor on live load (γL) of 1.6 or 1.4, as applicable; or
- The bridge element under consideration theoretically satisfies the nominated railway loading, however, based on a load factor on live load (γL) less than the preferred value of 1.6 or 1.4, as applicable.

The Principal Track and Civil Engineer may approve the reduction of the live load factor (γL) to less than the preferred value of 1.6, if a high degree of control and monitoring of the actual live load on a bridge is considered.

C2-8 Rating Results

Rating results shall be expressed as the ratio member capacity over applied load.

They shall be tabulated for “as new” and “as is”, and with and without full DLA.

Vehicle types and the effect of any speed restrictions in force or proposed shall be shown.

Where the rating is less than unity (1.0), the following shall also be included:

- Reduced speed necessary to raise the rating to unity (1.0), i.e. reducing DLA with respect to lower speed;
- Calculated load factor for live load with full DLA.

The results of any fatigue analysis shall also be provided.

A typical layout for the presentation of the rating results is shown in Appendix 1.

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.

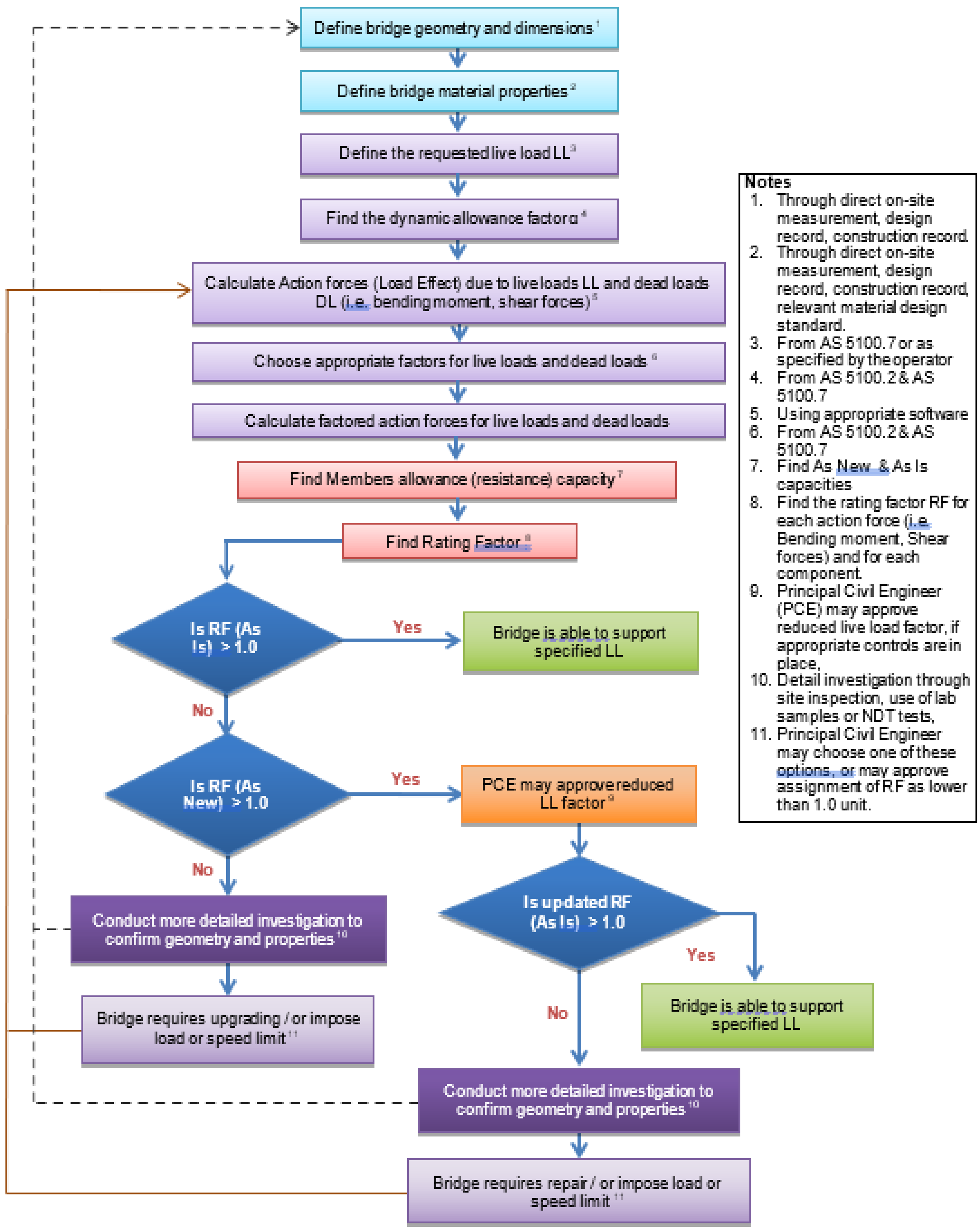


Figure 1 - Flowchart for rating process

Chapter 3 Investigation and inspection

C3-1 Desk 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 Loadings

C4-1 Changes in design loads

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

1. Cooper E (imperial)
2. Metric Cooper M
3. 300-A-12

4. 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:

C4-1.1 ANZRC Railway 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 2.

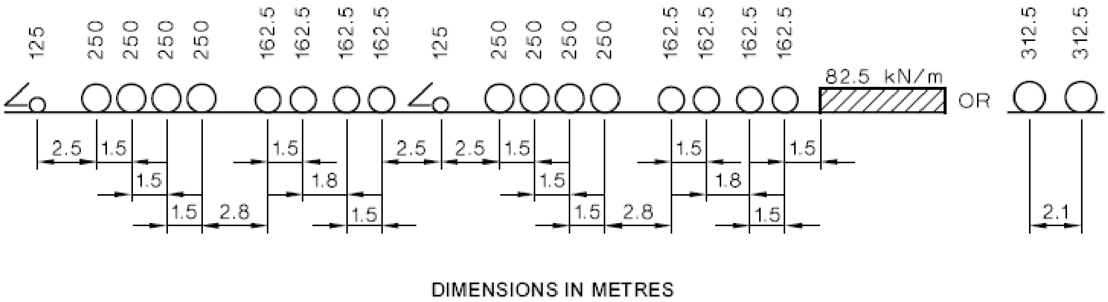


Figure 2 - M250 Live Load

C4-1.2 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 3.

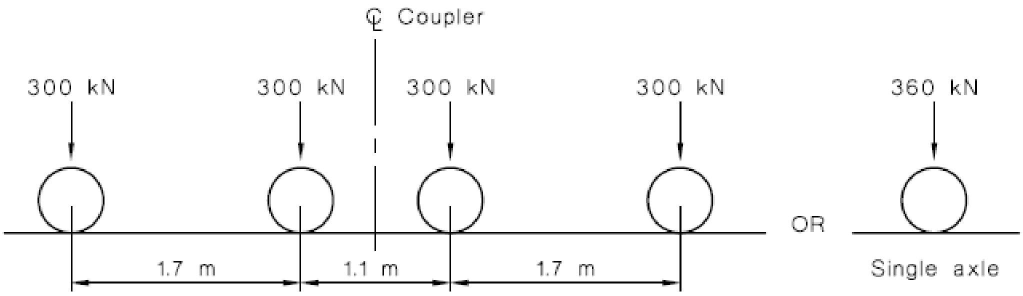


Figure 3 - 300-A-12 Axle Loads

The spacing between the centres of each axle group should be taken as 12m (see Figure 4).

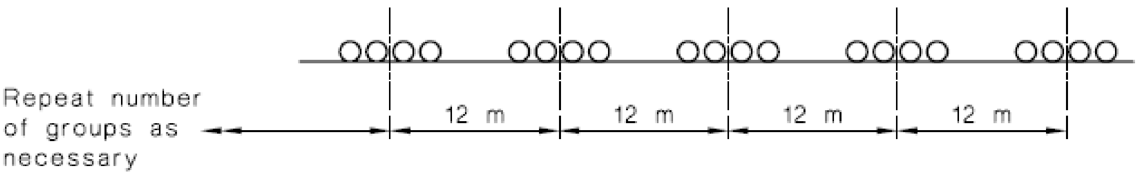


Figure 4 - 300-A-12 Axle Group Spacing

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.

C4-1.3 AS 5100 Bridge Design code (2004)

Figure 5 shows the 300LA loading which is the design load from the current bridge design code.

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.

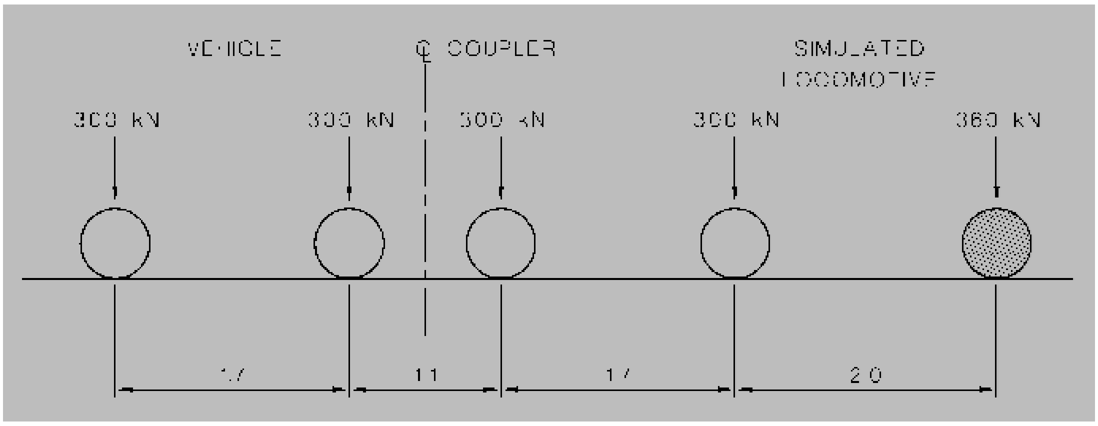


Figure 5 - 300LA Railway Traffic Loads – Axle Loads

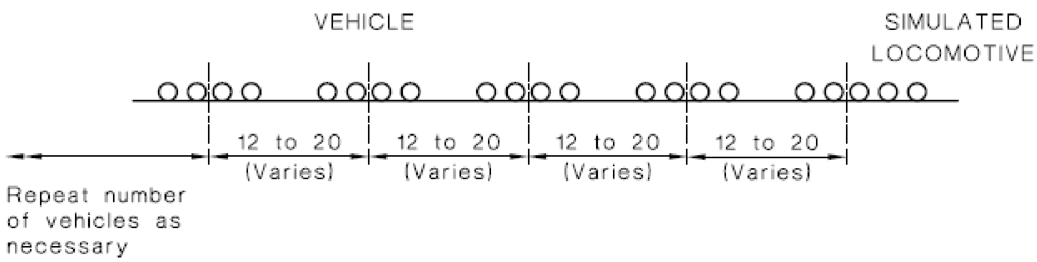


Figure 6 - 300LA Railway Traffic Loads – Axle Group Spacing

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:

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

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 current minimum design loads for the various lines are as follows:

The following design requirements are extracted from CRN Engineering Standard CRN CS 310 "Underbridges"

Track Class	Design Load configuration
Main Lines	
Heavy Haul Coal Operations	350-LA plus DLA
Class 1 and 2 lines	300-LA plus DLA
Class 3 and 5 lines	280-LA plus DLA
Sidings	
General Yard	300-LA plus 50% DLA
Sidings (includes unloading bins)	330-LA plus 0% DLA
Passenger operations/ or maintenance	180-LA plus 0% DLA

C4-2 Loads and loading factors

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

C4-2.1 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.

C4-2.2 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 in Appendix 2.

- 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

C4-3 Comparison of 300LA 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.

C4-4 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.

C4-5 Choice of load factors

C4-5.1 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, e.g. 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 (refer section C4-4).

C4-5.2 Load factors

Load factors for dead loads and railway traffic shall be in accordance with AS 5100 Part 7 (Table 7.3).

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.

C4-6 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. Thus in assessment of bridges for a speed greater than 80km/hr, the dynamic load allowance is the same as that for 80km/hr.

C4-7 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).

C4-8 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 5 Rating steel and wrought iron underbridges

C5-1 Rating requirements

For the superstructures of steel and wrought iron underbridges, 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.

C5-2 Steel Underbridges

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

S^* (Design Action Effect) < ϕR_u (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.

and R_u is dependent on material yield strength and geometry.

C5-3 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 1 - 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.

C5-4 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 2.

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

Table 2 - 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.

C5-5 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.

C5-6 Wrought iron and cast iron underbridges

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 comment on possible need to reduce ϕ for certain poor quality wrought irons.

C5-7 Inadequate load capacity under existing conditions

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

Traffic Conditions for main lines

- Train configurations documented in Section C4-2.2 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 13822cannot be attained then the load carrying capacity of that element shall be carried out using the dynamic load allowance from AS 5100.

Chapter 6 Rating concrete underbridges

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

S^* (Design Action Effect) < ϕR_u (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.

And 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 underbridges.

Chapter 7 Rating timber underbridges

C7-1 General

There are no references to timber bridges in AS 5100. Timber underbridges shall be rated using limit states methods in accordance with AS 1720.1 – "Timber structures Part 1: Design methods".

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

C7-2 Standard timber underbridges

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

C7-2.1 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 3 - Timber Girder Underbridge Superstructure Types

C7-2.2 Substructure types

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

Table 4 - Timber Girder Underbridge Substructure Types

C7-3 Rating parameters

Timber underbridges shall be analysed using parameters based on AS 1720.1 – “Timber structures Part 1: Design methods”, as modified in the following sections.

C7-3.1 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 in Table 5 to the nominal loads on the structure.

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.

C7-3.2 Live loads

Railway load configurations used for assessment shall be as advised by the relevant railway authority, together with the load factors in Table 5 below.

The Dynamic Load Allowance (DLA) for timber underbridges shall be in accordance with Section C7-3.15.

The ultimate railway load design action is equal to: (1 + DLA) x load factor x action under consideration.

C7-3.3 Ultimate limit state load factors for timber bridge load rating

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 5 - Ultimate Limit State Load Factors

C7-3.4 Capacity factors (ϕ)

Values of capacity factor (ϕ) for calculating the design capacity of structural members (Rd) and structural joints (Nd) shall be taken from AS 1720.1 Tables 2.1 and 2.2, Category 3 (primary structural members or joints in structures intended to fulfil an essential service or post disaster function).

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 1720.1 Tables 2.1 and 2.2 Category 1 (secondary members in structures other than houses).

C7-3.5 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 1720.1 Table H2.1.

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

The relevant portion of AS 1720.1 Table H2.1 is replicated in Table 6 below, with notes as follows:

- The characteristic values in Table 6 for bending apply for 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 6 by (300/d)0.167, where ‘d’ is the depth of the section.
- The characteristic values in Table 6 for tension apply for 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 6 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 6 - Characteristic Values for F22 Stress Grade Timber (MPa)

C7-3.6 Duration of load factor k1

Values for the duration of load factor k1 for the strength of timber shall be as follows:

k1 = 0.57 for permanent actions e.g. dead load, superimposed load, loads due to earth pressure

k1 = 0.97 for ultimate live load

Values for k1 for the strength of joints with laterally loaded fasteners shall be as follows:

k1 = 0.57 for permanent actions e.g. dead load, superimposed load, loads due to earth pressure

k1 = 0.86 for ultimate live load

Note that in accordance with Clause 2.4.1.1 of AS 1720.1, for any given combination of loads of differing duration, the factor k1 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 k1 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 k1 of 0.57 for permanent actions.

C7-3.7 Temperature factor k6

For the assessment of timber underbridges in New South Wales, the temperature factor (k6) shall be taken as 1.0.

C7-3.8 Strength sharing factor k9

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

C7-3.9 Modification factors k4, and k12

Modification factors k4 (partial seasoning factor) and k12 (stability factor) shall be in accordance with AS 1720.1.

C7-3.10 Round timbers

Where round timbers are used (such as in pier trestles or girders), these shall be assessed in accordance with Section 6 of 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 Clause 6.4.2. The shaving factor k21 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, k21 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.

C7-3.11 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.

C7-3.12 Girder composite action

Assume that double girders do not act compositely even in the case of underbridges 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.

C7-3.13 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 7).

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 7 - Continuity Corbel Effect

Note: ‘Single’ denotes a single span timber underbridge

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

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

C7-3.14 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.

C7-3.15 Dynamic Load Allowance (DLA)

Dynamic load allowance shall be determined as follows:

DLA = 0.05 V0.5 where V is the speed in km/hr (See Table 8).

Speed (km/hr)	DLA
20	0.22
40	0.32
60	0.39
80	0.45
100	0.50

Table 8 - Dynamic Load Allowance

C7-3.16 Centrifugal force factors

The centrifugal force factors shall be as follows:

Superstructure

$c = V^2/75R$ for transom top.

$c = V^2/150R$ for ballast top.

Trestle Bracing

$c = V^2/127R$.

Trestle piles

$c = V^2/70R$ for transom top with trestle height between 4m and 8m.

$c = V^2/220R$ for ballast top with trestle height between 3m and 6m.

V is the speed in km/hour and R is the track radius in metres.

C7-3.17 Soil pressure

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

$P = K_a \chi h$ with $K_a = 0.5$,

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

C7-3.18 Live load surcharge pressure

The live load surcharge pressure behind sheeted abutments at increasing depth in fill due to railway loading shall be computed in accordance with Clause 13.3 of AS 5100.2. It is noted that the draft version of AS 5100.2 contains a graph of vertical unfactored pressure versus depth below sleeper for 300LA railway load.

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

C7-3.19 Reference load

All rating data shall be derived from calculations based on 300LA railway traffic load.

C7-3.20 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.

C7-4 Non-standard rating parameters

All standard timber underbridges 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 8 Rating masonry arch underbridges

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 , $\gamma_{gs,}$ 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; and
- 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

(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 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. Refer to Figure 7.

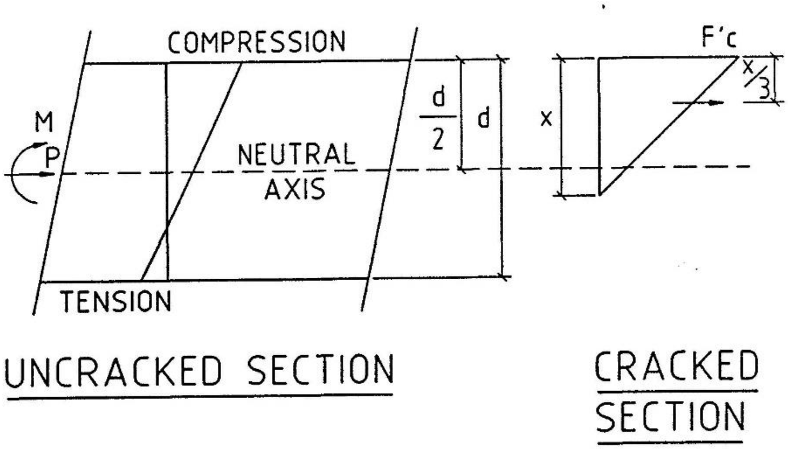


Figure 7 - Masonry Stress Diagrams

'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 9 Field testing

C9-1 Load testing

Determination of the live load capacity of a bridge can also be by 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 is 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 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 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.

C9-2 Strain gauging

C9-2.1 General

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

C9-2.2 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.

C9-2.3 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. Check transom top underbridges for effect of localised bending and torsion due to eccentric loading from transoms.

C9-2.4 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 rails and areas where cracks, loose rivets or broken bolts have occurred.

- For through girder underbridges, 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 trains 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 predicted dynamic load.

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

C9-2.5 Loading for strain gauging

A captive train (made available for the full testing regime) equal to or close to the maximum loading to be used on the underbridge 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.

C9-2.6 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.

C9-2.7 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 adequate 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. Further discussion of identification of wrought iron is found in CRN CM 305.

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 intersect 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 delaminations, 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 nick bend test has more than 50% crystalline fracture, consideration of rating using 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 delaminations.

Where samples are to be tested for yield and 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 is 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 delaminations. 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. Thus 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 BFBs 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. 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 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 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 much worse situations probably exist. Firstly, the weld probably will open some laminations as delaminations 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 reported on.

4. Welded steel girders prior to 1966.

Welded steel girders from 1966 onwards in underbridges 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 to be unweldable, unless proved otherwise by weldability tests. Steel produced up to 1925 can be considered to be even more unweldable. It should be noted that girders in jack arches will be in this category. Welds on these unweldable 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 fracture 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 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 underbridges

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

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

Collision damage from derailments should be 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 be from design or from fabrication. Examples are coping at the end of stringers or girders cut square with no radius; rough oxy cut surfaces; or 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 hot cracking.
- Fatigue cracks.
- Ductile tearing cracks. These are usually 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.

A1-4 Bracing systems

A1-4.1 General

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

A1-4.2 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 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.

A1-4.3 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.

A1-4.4 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.

A1-4.5 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.

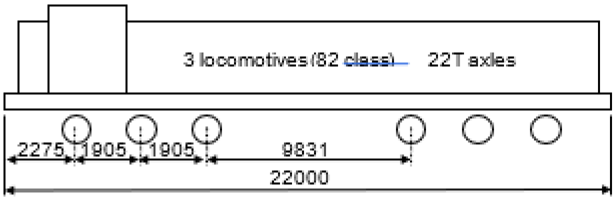
In most cases, bracing that was designed for welding when the bridge was new will perform much better than riveted bracing that has been repaired or strengthened by welding. This results in welds with a high incidence of HAZ cracking, and micro-cracking at the weld fusion zone. Much more rapid fatigue cracking will result, or even brittle fracture 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 Appendix 2 Loading Diagrams

MF loading



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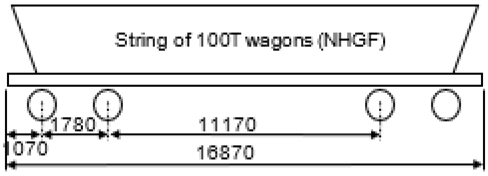
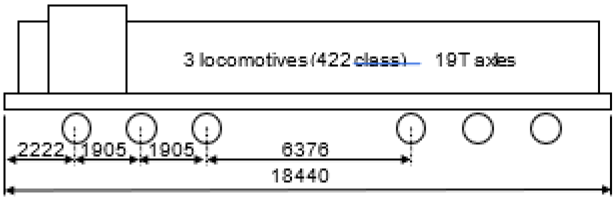


BF loading



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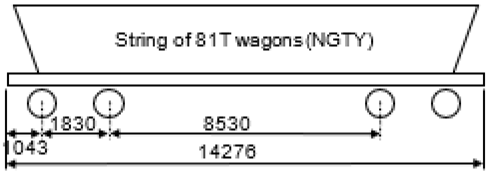
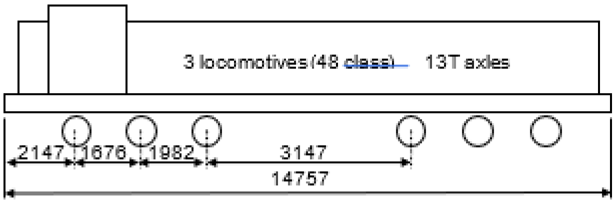


LB loading



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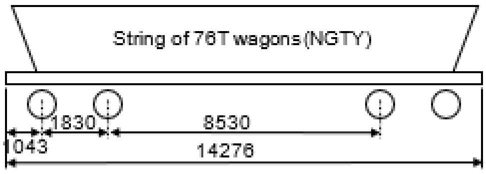
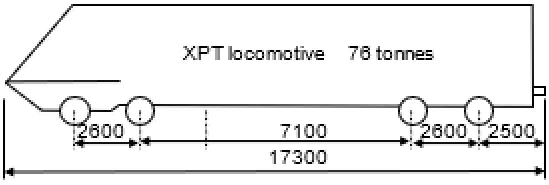


XP loading



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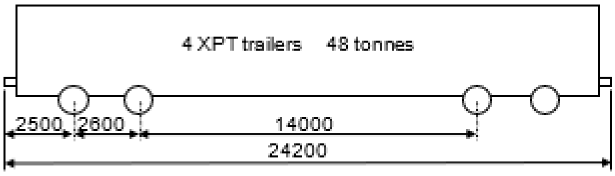
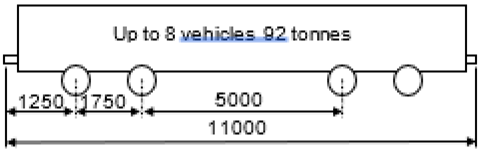


SB92 loading



Appendix 3 Presentation of rating results

Executive summary

Bridge Superstructure Member Rating (Speed > 80km/Hr)			
Main Long. Girder	Primary X Girder	Secondary X Girder	Secondary Long. Stringer
3.13	1.10	1.14	1.01

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

<table>
<tr><th colspan="6">Superstructure Connection Rating (Speed > 80km/hr)</th></tr>
<tr><td rowspan="3">Primary X Girder
To
Main Box Girder
(Bolts)</td><td rowspan="3">Long. Stringer
to
Primary X Girder
(Rivets)</td><td colspan="4">Secondary X Girder to Main Box Girder</td></tr>
<tr><td colspan="2">Complete Connection</td><td colspan="2">One Failed Web Cleat</td></tr>
<tr><td>Rivets</td><td>Cleats</td><td>Rivets</td><td>Cleats</td></tr>
<tr><td>5.11</td><td>1.08</td><td>1.06</td><td>1.11</td><td>1.04</td><td>1.01</td></tr>
</table>

Appendices

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