*STRUCTURES ENGINEERING*

# BRIDGE BRANCH
# DESIGN INFORMATION

**BRIDGE BRANCH**
**DESIGN INFORMATION MANUAL**

This Manual is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

## Authorisation

As head of Structures Engineering of Main Roads Western Australia, I authorise the issue and the use of this Manual.

**A. LIM**

SENIOR ENGINEER STRUCTURES

Date : 06/07/2022

**Document No: 3912/02-1**

**This document is only controlled via the Main Roads website**

# BRIDGE BRANCH DESIGN INFORMATION

**SECTION 1**

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| 2 | 1 | 25/10/04 | Index sheet amended. Section 15 re-titled and Section 16 incorporated into Section 15. |
| All | 2 | 12/12/05 | Complete review for introduction of AS 5100 |
| 2 | 3 | 22/01/08 | Index sheet amended. New Section 16 added. Update clause 1.2. |
| 2 | 4 | 18/06/09 | Index sheet amended. New Section 18 added |
| All | 5 | 03/04/18 | Complete review for introduction of AS5100-2017 |
| 2 | 6 | 06/07/22 | Index sheet amended. Section 4 re-titled |
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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 06/07/2022

# 1 INTRODUCTION, SCOPE AND PURPOSE

## 1.1 Introduction

The Bridge Branch Design Information Manual has been prepared to provide guidance and to set design criteria for the process of carrying out design and design related activities for bridges, culverts and other transport related structures. The Manual presents information and criteria for use in structural design to assist with the design process, clarify code ambiguities and to ensure uniformity, consistency and conformity of results.

The Manual has been prepared in good faith. However, Main Roads Western Australia (MRWA) does not guarantee or warrant the veracity of any information or referenced information contained within. The use of information and criteria contained within this Manual shall not relieve the user of their responsibilities for due diligence and checking of all results and outcomes.

## 1.2 Scope

The scope of the Manual includes, but is not limited to, the following:

- Design of new road and pedestrian bridges;
- Design of new culverts;
- Refurbishment design of existing bridges, including strengthening;
- Criteria for
  - Clearances
  - Bridge width
  - Bridge design loading
  - Serviceability stress limits
  - Construction forces and effects;
- Load rating of existing bridges, including historical design and load rating vehicles;
- Bearings and joints (including approach slabs);
- Railings and barriers; and
- Waterways investigation and flood estimation.

Unless specifically excluded by an authorised officer from Structures Engineering, all structural design undertaken within Main Roads Western Australia or by its authorised Agents shall incorporate the guidelines, methodologies, processes and criteria presented in this Manual.

## 1.3 Purpose

The principal purpose of this Manual is to:

- ensure uniformity of standards and details in the design of bridges, culverts and other transport related structures;
- record any variations from AS 5100 Bridge Design (CODE) approved for use in the design process;
- clarify any confusing or ambiguous areas of the CODE; and
- ensure that construction feedback receives widespread circulation.

Any person identifying a need for any of the above not already covered adequately elsewhere shall notify the Structures Design & Standards Engineer, who shall arrange the writing and issuing of any required revisions, if considered appropriate.

## 1.4 Use of this Manual

In using this Manual, references to other information Sections within the Manual are in the following format:

Design Information Manual / Document Number / Section Title

The above format is abbreviated to DIS 3912/02/xx "Section Title", where DIS represents 'Design Information Section' and 'xx' represents the particular Section within the Manual.

## 1.5 Common Abbreviations

Common abbreviations used throughout this Manual are:

- DIS - Design Information Section
- BDC or CODE - Bridge Design Code, AS 5100
- MRWA - Main Roads Western Australia
- SES - Senior Engineer Structures
- SD&SE - Structures Design & Standards Engineer
- EBL - Engineer Bridge Loading

# SECTION 2 – DESIGN OF NEW STRUCTURES

This information is Section 2 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As the head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 01/10/2024

**Document No: 3912/02-2**

**Controlled Copies shall be marked accordingly**

**SECTION 2**

**CONTENTS**



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 09/12/05 | Complete review for introduction of AS 5100-2004 |
| Appendix C | 2 | 04/05/06 | Amended Design Summary Sheet – Sample Only - Sheet No. 2 of 2 |
| 5, 6 and 8 | 3 | 11/12/06 | Collision from railway traffic updated, RTEM guide renamed and use of Polymeric material for MSE walls |
| Appendix A | 4 | 02/04/12 | Amended Design Summary Sheets – Load Rating – Group 2 Vehicles 4 & 5 Non Supervision added |
| All | 5 | 04/09/23 | Full review and alignment with SWTC |
| All | 6 | 01/10/24 | Minor update and corrections |
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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 01/10/2024

# 2 DESIGN OF NEW STRUCTURES

## 2.1 Introduction

The design of new structures shall be carried out in accordance with generally accepted engineering principles in conjunction with AS 5100 Bridge Design (Code). Additions and modifications to the Code are required as per Section 2.5.

General design guidance is provided in the Structures Engineering Design Manual, (Document No. 3912/03).

Main Roads staff may access the design process guidelines and report template provided in the Structures Engineering Management System manual (Document No. 3912/01).

Stakeholders should understand that Structures Engineering provides advice, not approval or direction. The Engineer is fully responsible for the design and the Project Manager is fully responsible for risk.

## 2.2 Preliminary Design

Great importance is attached to carrying out adequate Preliminary Design to ensure that all options and constraints are identified so that the most appropriate design can be selected. The potential for improvement by appropriate selection of structure type and construction method is much greater here than in final design, where only marginal refinements are possible.

This concept stage is vital and both experience and imagination are required. Sufficient time and effort <u>must</u> be put in before beginning the detailed design to ensure the best option is selected in terms of form, function, aesthetics, constraints such as site conditions, practicality and cost of both construction and maintenance.

During the preliminary design stage it is important to consider all constraints as these will have a considerable influence on the chosen structure type and components. More information on common constraints can be found in Section 2 of the Structures Engineering Design Manual (Document No. 3912/03), but amongst the more important constraints are:

- clear spanning requirements
- any staging requirements for future road or rail widening
- structure type, product selection and preventative treatments to minimise whole of life cost or improve value
- access for inspection (from parking to visibility of key components)
- environmental and/or ethnographical considerations
- need to maintain traffic, rail or navigation clearances during construction
- choice of construction method (difficult site conditions, restricted access etc)
- influence and importance to be placed on aesthetics

The preliminary design process is a simplification of final design. The aim is to obtain an approximate idea of member sizes, reinforcement, foundations etc so that a reasonably accurate cost estimate can be prepared. The amount of effort put in at this stage will obviously depend on the size, importance and cost of the particular structure.

It is normal to produce at least General Arrangement and Cross Section drawings of each bridge option or scheme investigated. Other drawings or sketches may be necessary so that an approximate Bill of Quantities can be prepared to enable an estimate to be calculated.

On completion of the preliminary design, the Design Engineer is required to produce a 15% design report and recommend an option to progress to detailed design. When recommending an option it is important that the Design Engineer not make assumptions about what price will be acceptable to Main Roads. It is up to Main Roads staff to obtain the funding required, or to select a different option if funding of the ideal option has not been achieved.

A simplified flow chart of the Preliminary Design process is presented in Appendix A.

## 2.3 Prior to Detailed Design

Prior to detailed design it is required that gaps in standards be identified and additional clauses/alternative standards to be used be submitted for review with comprehensive and unbiased research, optioneering and justification. Depending on scope and budget, this can be done as part of the 15% process or at the beginning of the detailed design.

## 2.4 Detailed Design

Detailed design is a natural progression of the preliminary design. All bridge and structure design shall be carried out in accordance with the Code, as modified by this Manual (refer section 2.8). The bridge designer shall also ensure that the transverse load rating is greater than the longitudinal load rating for any given design vehicle.

## 2.5 Design Criteria

The design criteria present the parameters used in the design. The design criteria is used to confirm accuracy of the design parameters prior to committing effort to detailed design. The design criteria may also be of use in future works if the asset is to be modified or load rated.

Main Roads has the following mandatory design criteria documents:

| | |
|---|---|
| Bridge Design Criteria | Criteria form to present the conceptual design. Requires SES signature before detailed design can proceed. |
| Bridge Specific Design Criteria | Bridge specific design parameters that are submitted for review with the design report. |
| Standard Bridge Design Criteria | The Standard Bridge Design Criteria has standard design parameters and code references. It exists so that these are not re-produced in every design report. Do not include this document in report appendices.<br>The Standard Bridge Design Criteria is not to be modified. Any authorised Deviation from standard shall be added to the Bridge Specific Design Criteria instead. |

The design criteria forms need to be filled out to an extent that is appropriate to the stage of design. It is acknowledged that some items will be unknown at early stages of design.

In cases where an item in the bridge specific design criteria is not applicable, "N/A" can be entered in place of a value. The need to offer an explanation is at discretion of the engineer but explanations do assist the reviewer/verifier. In example, for "Road Bridges – Vibration" the value for First mode flexural frequency could be entered as "N/A (compliant deflection)".

This design criteria forms can be used for an Approval in Principle process where they are submitted early for review. Most designs will not require this process.

In an effort to not repeat the items from the Bridge Design Criteria Checklist in the Bridge Specific Design Criteria, it is intended to include both forms in the structural report.

It is expected that the design report will include a table as follows:

| Type of Criteria | Description |
|---|---|
| Bridge Design Criteria | Refer Appendix X |
| Bridge Specific Design Criteria | Refer Appendix X |
| Standard Bridge Design Criteria | Main Roads Standard Bridge Design Criteria, dated dd/mm/yyyy |
| Relevant RFI | Include document number, subject and date. The author may decide whether they need to be included in an Appendix, generally this is not required unless an issue is contentious or not resolved. |
| Deviations from Standard | List here, eg: "Topic – Authorised" or "Topic – In progress".<br>Update the Bridge Specific Design Criteria to match.<br>Include SES authorisation in an Appendix. |

## 2.6 Required Interfaces

A summary of interfaces with Structures Engineering is presented below.

| Item/Form | Responsible role & Contact address | Notes |
|---|---|---|
| Information requests | DO structengreviews@ mainroads.wa.gov.au | |
| New bridge number | DO structengreviews@ mainroads.wa.gov.au | |
| Vehicle loads | EBL structengreviews@ mainroads.wa.gov.au | |
| Bridge Design Criteria | SES structengreviews@ mainroads.wa.gov.au | Must be submitted prior to detailed design |
| Bridge Specific Design Criteria | Review Team structengreviews@ mainroads.wa.gov.au | This will generally be submitted as part of the design report, except for special cases where agreement in principle is considered necessary prior to design |
| Departure from Standards | SES structengreviews@ mainroads.wa.gov.au | Must be submitted prior to detailed design |
| Waterways | SWE eric.cheung@ mainroads.wa.gov.au | |
| OS&H Report (Safety in Design) | Project Specific (e.g.: Project Manager) | Typically submitted as part of the design report. Not reviewed by Structures Engineering |
| Bridge/Culvert Inventory form | DO structengreviews@ mainroads.wa.gov.au | |
| Submissions for review | Project Specific (e.g.: Project Manager) | The PM will provide to Structures Engineering if review is required.<br>Drawings are not accepted without an associated design report and are not to be split into piecemeal packages.<br>Structural, durability, waterways, TQ and RFI reviews only |
| Design Summary Sheets and model files | EBL structengreviews@ mainroads.wa.gov.au | The DSS incorporating load ratings for all design vehicles must be submitted and get reviewed at IFC Design Stage. |
| As Constructed information | Project Specific (e.g.: Project Manager) | The PM is responsible for sending to Structures Engineering.<br>All X-refs to be bound. |

It is important to only submit quality work. Work that is not of suitable standard will be returned unactioned. When this occurs there is a mandatory two week period where the design and drawings are to be reviewed and updated before they may be sent again for review.

Do not include the email address of individual Structures Engineering staff in the email cc field when sending a request to structengreviews@mainroads.wa.gov.au. Doing so impacts internal document control processes and extends turnaround times.

## 2.7 Design Summary Sheets

Design Summary Sheets (DSS) shall be prepared by the Design Engineer for all new bridges, dual use path bridges with vehicle access and culverts/underpasses with clear spans over 3.0m. summarising the most important features of the design on completion of the design and independent design verification. The Design Summary Sheet is critical to load management as it contains a summary of all the major design components and is used to check heavy load movements and design future works. The actual contents will vary depending on the size and complexity of the structure. Typical Summary Sheet details for a simply supported structure and a continuous structure are attached at Appendix D and E respectively and should be adhered to as a minimum standard. The DSS incorporating load ratings for all design vehicles must be submitted at the IFC (i.e., Issued for Construction) stage to MRWA for review and acceptance by MRWA.

The Design Engineer MUST report the load rating factor of each new bridge for each nominated rating vehicle in accordance with AS 5100.7 and Section 4.

The main items to include on the Design Summary Sheet are:

- Details of the span configuration
- The design cross-sections used in the analysis at critical positions, e.g., support and midspan
- The section properties of these design cross-sections
- Details of the reinforcement and/or prestress and the section capacities at the critical sections
- The serviceability design moments and resulting stresses at the critical sections
- Live Load Distribution Factors for different loadings
- The available live load capacity at the critical sections, for use in checking heavy load movements
- Foundation information, i.e., design bearing pressures for spread footings, design pile loads for piled foundations
- Design scour allowance
- Load rating information for all design and rating vehicles in accordance with Section 4 of BBDIM

## 2.8 Code Modifications

There are three main sources of modifications to the Code:

1. SES Circulars contain clarifications or mandatory Code modifications for Main Roads projects. The Design Engineer must check for currency of SES circulars as they may have been recalled or updated. SES Circulars are available on the Main Roads website.
2. This Manual contains mandatory Code modifications for Main Roads projects. The entire Manual is relevant, although the reader's attention is drawn to Appendix B & C in this section.
3. Deviations from the Code as requested by the Design Engineer. Any deviations from the Code require written approval from Senior Engineer Structures. Past practice is not an acceptable reason to deviate from the code on new projects. As such, it is noted that all approvals from SES are project specific.

# APPENDICES

# APPENDIX A PRELIMINARY DESIGN PROCESS

**Step 1**: assess the site where the bridge is to be located. The site constraints will influence the bridge type, configuration and method of construction.

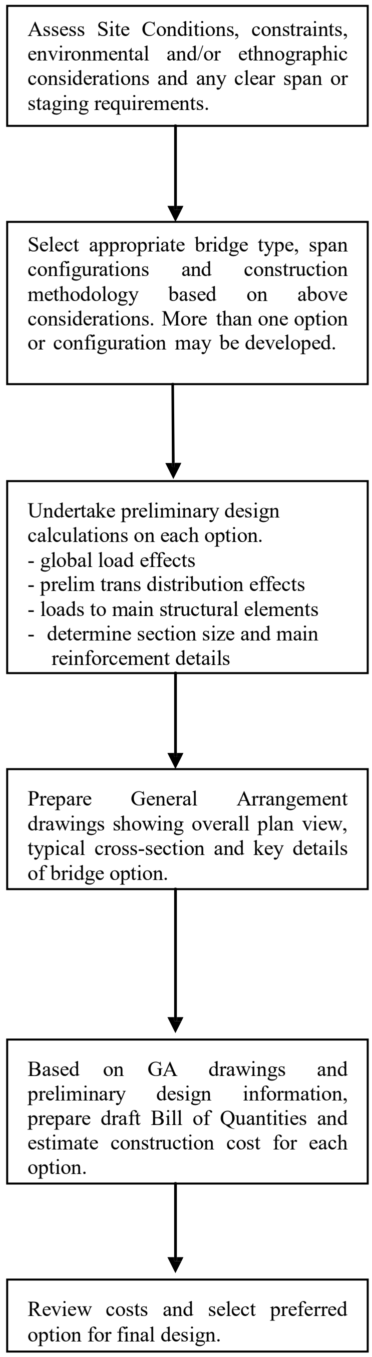
**Step 2**: an appropriate option for bridge type (reinforced or prestressed concrete, steel or composite), configuration (number and length of spans, bridge width), foundations (piled or spread footings) and construction methodology (in situ, precast, segmental, incremental launch).

**Step 3**: preliminary design can range from simple global models and simplified distribution analysis to use of grillage or FE models, depending on the complexity of the bridge.

**Step 4**: preliminary drawings showing general details of the overall bridge configuration, superstructure and substructure together with relevant sections and details will assist with estimating construction costs and visualising the bridge.

**Step 5**: construction costs are estimated (usually by a Quantity Surveyor) based on the details provided in the preliminary drawings, and are required to assess the economic attributes and cost-effectiveness of the various options.

**Step 6**: select the best option that meets all specified criteria to proceed to final design.



## APPENDIX B MODIFICATIONS TO AS 5100

The clause numbers below refer to clause numbers as used in the Bridge Code. For example: BDC 1 - 13.1 refers to Bridge Code Part 1, Clause 13.1.

### AS 5100.1 – SCOPE AND GENERAL PRINCIPLES

BDC 1 – 8.2 Design Life

Design life shall be in accordance with the Standard Bridge Design Criteria (D23#845709).

Art work and rebates must not compromise the design life of an asset.

BDC 1 - 13.6 Horizontal Clearance to Substructure Components of Bridges over Roadways

The face of any support, apart from columns located in the median, must be set back from the ultimate edge of the nearest traffic lane as per Figure 12.1 in Section 12.

BDC 1 - 13.7 Vertical Clearance of Structures

Refer to Section 12 for requirements.

Note that headroom requirements are presented in Appendix C.

BDC 1 - 14 Traffic Barriers

Main Roads' standard barriers that are deemed to comply with the respective Code Performance Levels should be used where possible.

Refer to Section 7 for additional requirements.

BDC 1 - 15.3 Collision from Road Traffic

For guidance on the protection of sign structures, see Austroads Guide to Road Design Part 6.

BDC 1 - 15.3 Collision from Rail Traffic

Whether deflection walls are required is dependent on assessment in accordance with the Code.

Where a sign structure has potential to fall and endanger life, the sign supports must be assessed in the same way as bridge supports.

BDC 1 - 16 Pedestrian and Bicycle-Path Barriers

Where a structure, such as a retaining wall, head wall or wingwall, presents a vertical or near vertical face 1.5 m or more in height and it would be likely that a person could gain access to the upper edge of the structure, an AS5100 compliant pedestrian restraint system shall be installed close to, or on top of the structure. Fences are not acceptable.

A risk assessment shall be carried out for the protection screen requirements for objects falling or being thrown from bridges in accordance with Main Roads' assessment procedure outlined in the "*Risk Assessment for Projectiles Thrown from Overpass Structures*" D22#878096. It is important to seek information from local police stations as there may be existing projectile rock throwing risk that would otherwise not be identified.

Protection screens (all types) must not be installed within the working distance of barriers.

## BDC 1 - 18 Drainage

For bridges that are not on a crest, upstream road infrastructure shall be used to intercept water before it crosses a Deck joint.

Drainage pipes are not preferred due to the difficulty of maintenance. They should only be installed where required for Environmental reasons such as discharging water away from protected water sources.

All bridge drainage pipes whether external or internal must be of durable material, must be corrosion and fire resistant, and must be concealed from public view where possible.

All drainage structures must be vandal proof and accessible for cleaning (eg: pressure blasting and rodding), operation and maintenance purposes.

## BDC 1 - 19 Access for Inspection, Maintenance and Component Replacement

Access shall be provided such that each component can be inspected and maintained (or replaced if applicable).

Where steps are used in areas subject to flood loading it is preferred to install them on the downstream side as they collect debris.

## BDC 1 - 20 Utilities (Services)

When a service is installed within the bridge structure, it shall be installed where it is safest and maintainable.

It is expected that:

(i) Services are installed in a footpath slab where possible. This includes having all spare conduits in one side of the bridge and crossing the road using off-bridge junction pits.

(ii) Services cast into the beams shall generally be avoided.

It is required that

(i) Water (including sewerage), gas and oil services shall not be located within a void unless agreed by the service authority.

(ii) Service types shall only be mixed where approved by the service owners. For example, water/gas/oil and electricity may require separation.

(iii) Services shall be concealed from view if possible.

(iv) Adequate provision must be made for future inspection, maintenance and possible replacement.

Spare ducts comprising two 100 mm (internal diameter) conduits and draw wire must be provided on each side of the bridge terminating with capped ends beyond the deck and approach slab.

Where a bridge is subject to flooding and the bridge type provides limited space to have cast-in conduits, it should be considered whether service protection against flooding is more important than the number of spare ducts. If there are no services crossing the bridge, then installing only one spare duct per kerb is generally preferred to hanging spare conduits. Allowance shall still be made to install the additional ducts in the future if necessary, eg: by providing penetrations for future conduits in abutment walls.

Services must not require access (including inspection and maintenance) via traffic lanes. Locations of pits and conduits must not conflict with traffic barriers.

**AS 5100.2 – DESIGN LOADS**

BDC 2 – 7 Road Traffic

For additional requirements refer to Section 5, Design Vehicle Loadings.

BDC 2 - 6.3 Superimposed Dead Load

Traffic barriers, surfacing, parapets and any non-structural components must be treated as superimposed dead load.

Road bridge structures with an open graded asphalt wearing surface must be designed for the load from a total asphalt thickness of 100 mm. All other road bridges must be designed for the load from asphalt surfacing of 75 mm thick. Main Roads bridges on a highway or main road (ie. M or H class road) may take the surfacing super imposed dead load as "controlled".

Bridge geometry such as deck levels and the depth of concrete upstands shall depend on the seal type. Refer to 3912/02/08 "Bearings and Joints" for general guidance.

BDC 2 – 7.3 Heavy load platform

For additional requirements refer to Section 5, Design Vehicle Loadings.

BDC 2 – 7.5 Standard design lanes

Use b = width (in metres) between external barriers. Where the barrier has an adjacent kerb this width may be subtracted during calculation of b.

If it can be demonstrated that there will be no future need to replace a median skylight with trafficable surface, then the design lanes may be placed to avoid the skylight. Doing so requires Structures Engineering and Road and Traffic Engineering approval.

All other traffic barriers, balustrade, medians, footpaths, must be treated as usable carriageway and their width included in the calculation of b.

Where there is no traffic barrier separating the carriageway and paths or raised verges, the Deck must be designed for SM1600 loading between traffic barriers with path concrete in place.

For additional requirements regarding vehicle positioning refer to Section 4, Load Rating of Bridges.

BDC 2 – 14 Earth Pressures for Integral and Semi-integral Bridges

Earth pressures that develop behind integral abutments and end screen walls may be calculated as per PD 6694-1:2011. In this case the load factor for passive earth pressure must be as per Table 6.4 of AS5100.2.

BDC 2 - 22 Construction Forces and Effects

In addition to Code requirements, the following criteria shall apply during the construction stage:

(a) Construction Live Load of 1.0 kPa minimum on entire surface

(b) The design differential settlement shall allow a component for settlement during construction (permanent effects and construction live load) of no more than 25mm.

For launched bridges, a note is required in the construction specification that launching is not to be carried out during strong winds.

## AS 5100.3 – FOUNDATIONS AND SOIL-SUPPORTING STRUCTURES

BDC 3 - 5.3.1 Shallow Footings

The use of sill beams (beam-like shallow foundations) is not permitted on road bridges. Sill beams are permitted on footbridges provided there is a minimum footing embedment of 1.5m.

BDC 3 - 6 Piled Foundations

Continuous Flight Auger (CFA) piles will not be permitted:

(A) in cohesive soils, silts or soil profiles with layers of coarse gravels or larger particles, except where excavation of an uncased hole near vibration sensitive Services is not possible;

(B) where the concrete exposure classification is more severe than B1 in accordance with Section 5 of the Bridge Code;

(C) where a socket in rock of a better quality than highly weathered is required;

(D) where a rock socket longer than 300 mm is required;

(E) where raked piles are required;

(F) where the soil profile is complex with hard layers over soft layers; or

(G) for end bearing piles, where the bearing stratum is on a slope steeper than one vertical to four horizontal.

## AS 5100.4 – BEARINGS AND DECK JOINTS

Design shall be in accordance with Section 8, “Bearings and Joints” and as follows:

BDC 4 - 5 Functions of Bearings and Deck Joints

The bridge deck slab shall be continuous between abutments.

BDC 4 - 12.1 Elastomeric Bearings - General

Where the use of elastomeric bearings is proposed bearings must be selected from Appendix A of the Code.

BDC 4 - 12.6.7 Fixing of Bearings

Bearings must be restrained in position by recessed pockets in concrete Substructures or by mechanical devices.

BDC 4 - 19 Deck Joints

Joints that are primarily rubber that incorporate embedded steel plates shall not be used.

Joints must not inhibit the proper placement of concrete.

For new bridges, where the deck joint is attached by bolts cast into a concrete substrate, fully tensioned high tensile bolts must be used. Where the deck joint is attached by bars cast into concrete, flat bars must be used. Alternatively, proprietary joints with documented evidence of long term performance may be submitted for to SEB for authorisation. The evidence submitted shall not be reliant on manufacturer brochures.

### BDC 4 - 19.5 Drainage for Road Bridges

Deck joints must be designed to ensure the joint will contain run-off over the full width of the Deck, including at paths. A maintainable backup drainage solution should be provided if possible.

Prefabricated extrusion joints must be turned up directly behind the carriageway kerbs to maintain the same installation depth from the Deck or Superstructure surface, and extended for the full height of the kerb to contain runoff. Deck joints in shared path bridges or across raised parts of the Deck or Superstructure including paths, medians and parapets must be concealed and protected using a recessed steel cover plate. Deck or Superstructure crossfall must be designed to prevent water leakage or spilling from the Deck or Superstructure at the joints.

## AS 5100.5 – CONCRETE

The following requirements are applicable to both superstructure and substructure design.

### BDC 5 – 1.2 Application [Reinforcement]

Ductility Class L and/or LP reinforcement shall not be used for strength, except in culvert base slabs, apron slabs, and walls up to 2 m height.

### BDC 5 – 4.14.3 Cover for Corrosion Protection

(A) The Designer must identify whether standard formwork and compaction or rigid formwork and intense compaction applies. It is noted that not all faces of precast elements receive intense vibration, particularly beam top and bottom flanges. Where rigid formwork and intense compaction is used, unless specifically allowed by Main Roads Specification 820, the engineer will note on the drawings that self-compacting concrete is not permitted.

(B) Where curing compounds are used, the cover shall be increased by 5 mm for classifications A and B1, and 10mm for other classifications.

### BDC 5 - 4.3 Exposure Classification

Protective surface coatings must not be taken into account in durability design or assessment.

### BDC 5 - 4.4.2 Curing

Accelerated curing by methods other than steam curing will not be permitted.

BDC 5 – 8.1.7 Stress in bonded tendons at ultimate strength

The maximum stress reached in bonded tendons at ultimate strength ($\sigma_{pu}$) shall be calculated based on the stress-strain relationships shown in Figure 2.1.

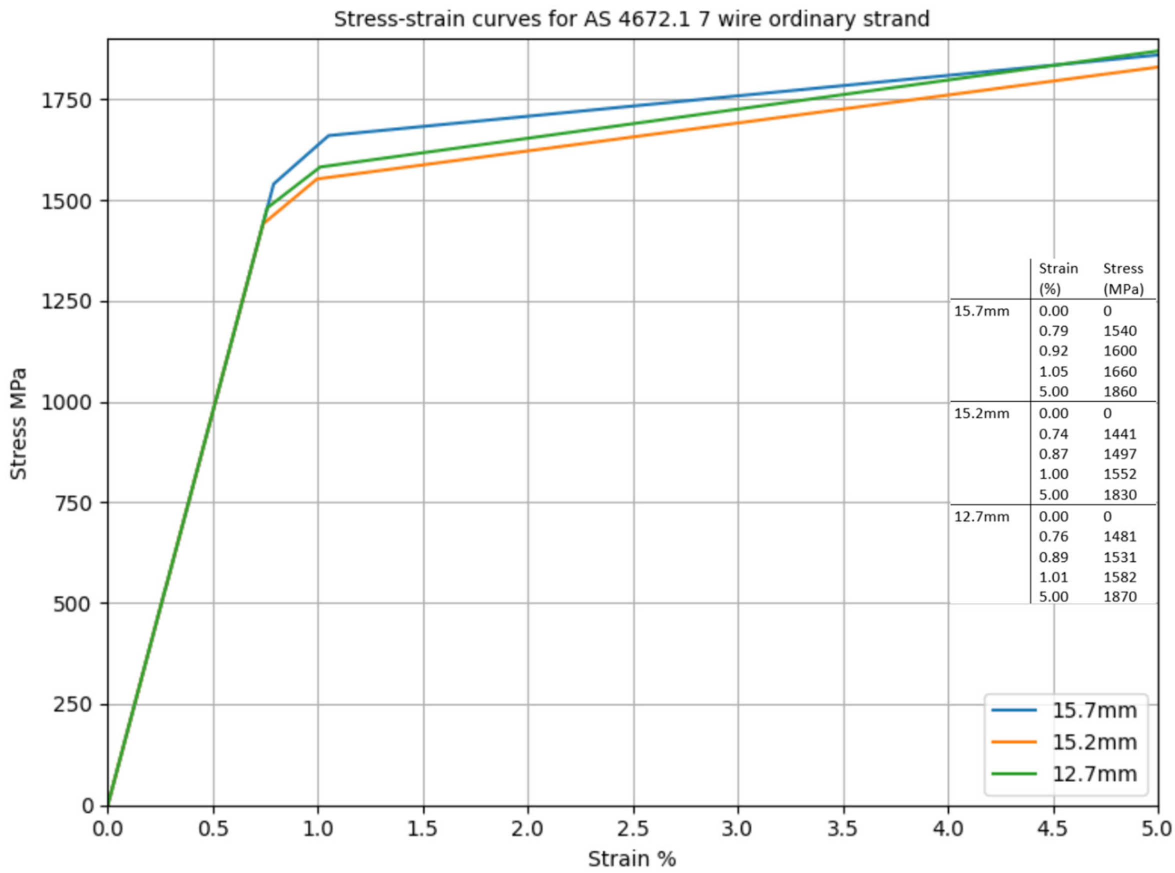



Figure 2.1 Idealized Stress-Strain Curve for 12.7, 15.2 and 15.7mm strand

The idealised trilinear stress-strain curves (Figure 2.1) are based on tensile properties of strand as per AS 4672.1 with a design breaking strain of 5%.

BDC 5 – 8.2 Strength of beams in shear and torsion

Section 4 provides a detailed example for using an iterative MCFT method to calculate the capacity of prestressed concrete members. Note the values of capacity reduction factor, $\Phi$, differs depending on the failure mode.

Please note, the approach presented in Section 4 to use of z as the internal moment lever arm as an approximate method should NOT be used in design or load rating for the Design Vehicles. Rather, the Design Engineers should use the known actual additional longitudinal reinforcement used for shear and torsional demand. Also, for the purpose of independent design verification and future calculation, the additional reinforcement shall be clearly demarcated in the relevant design drawings.

For determining if torsional strength should be considered or not as per Cl. 8.2.1.2 AS 5100.5, T* shall be calculated from both cracked and uncracked sectional analysis as explained in Section 4.

BDC 5 – 10.7.4.2 Restraint of longitudinal reinforcement

The detailing in item (a)(iv) is only acceptable as restraint of longitudinal column reinforcement. Fitments can only be used for strength if complying to the requirements of AS 5100.5 Clause 8.3.2.4

BDC 5 – 13.2 / 13.2.5 Splicing of reinforcement

Bundled bars must be accounted for in determining lap length. Bundling caused by splicing of reinforcement must be included.

# APPENDIX C OTHER STRUCTURAL DESIGN REQUIREMENTS

Structural design requirements that do not relate to a specific clause in AS 5100 are listed below.

1.Minimum Headroom

To assist in safe future inspection and maintenance, bridge clear headroom shall not be less than 1.0m.

2.General Reinforced/Prestressed Concrete Requirements

The use of half joints requires SES approval.

3. Precast Prestressed Concrete Beams

If precast prestressed concrete trough beams are used for the Superstructure, then the following detailing requirements must apply in the design and manufacture of beams:

(A) Only open trough type beams will be permitted

(B) Internal diaphragms must be provided at each end of each beam

(C) Intermediate internal diaphragms must be provided at a maximum spacing of third points along the beam

(D) Unless alternative measures are implemented to cater for torsional effects, cross girders / external diaphragms must be provided at the ends of each span between beams and must comply with clause 3.6(b)(xiii)

(E) Web thickness must be a minimum of 125 mm to assist placement and compaction of the bottom flange concrete

(F) The maximum bridge skew must be 30 degrees

(G) Cover to the top face of the top flange must be a minimum of 20 mm

(H) In order to prevent the development of intersecting horizontal cracks between the strands, not more than 50% of the prestressing strands must be debonded at any section, including beam ends. Debonding of adjacent strands is only permitted where there is at least 100 mm of concrete between the strands.

4.Reinforced Soil Walls

Refer to specification 802 for other design requirements.

MSE Abutments will not be permitted on structures over a water course.

The MSE panels must extend to the underside of the superstructure with minimum clearance sufficient to enable the maintenance and replacement of the bearings.

MSE wall panels are to be full height except segmental panels are acceptable for bridges over rail.

The top of all MSE walls and the area adjacent to the Abutment, including associated wingwalls, must be stone pitched for 1.0 m width to prevent ingress of water into the reinforced soil block. Bituthene, or equivalent membrane, must be used to prevent loss of MSE wall backfill through facing panel joints.

## 5. Abutments and Piers

Abutments must be constructed so that any spill through embankment, backfill, or Abutment footings can be removed in the future down to 200 mm below the nearest road shoulder level without reducing structural adequacy.

Abutments and Piers must be of concrete construction. Steel piers are acceptable for Shared path bridges. Facing panels will not be permitted.

Minimum requirements for footings and pilecap soffit levels:

- On bridges over permanent water, 300 mm below design scour level and lowest astronomical tide
- On bridges not over permanent water, 300 mm below finished ground level. Note that AS5100.1 clause 15 requires this minimum be increased to 1.2m where a pier is functioning as a collision wall, however this shall be taken to apply to spread footings only.

## 6. Approach Slabs

Bridges longer than 30m shall have approach slabs. For bridges between 20 and 30m an allowance shall be made to install future approach slabs by incorporating a suitable corbel into the skirt beam or abutment curtain wall.

Approach slabs must be of uniform length and must be at least 5.0 m long measured normal to the expansion joint.

## 7. Waterproofing

The waterproofing design must be in accordance with Main Roads' Specification 875, *Waterproof Membrane*.

## 8.Shared Path Underpasses

(i) All underpasses must be single span concrete structures. Cladding will not be permitted. The Deck cross-section of shared path underpasses must have a continuous flat or curved Soffit, that is, Discrete Beams will not be permitted. Steel structures will not be permitted.

(ii) Underpass Skylights shall be designed for W80 wheel load at any location. The structure that supports the skylight may be subject to traffic loads as per Main Roads amendments to BDC 2 – 7.5.

## 9. Culverts

Culvert barrels must follow the natural waterway alignment.

## 10. Future Modifications

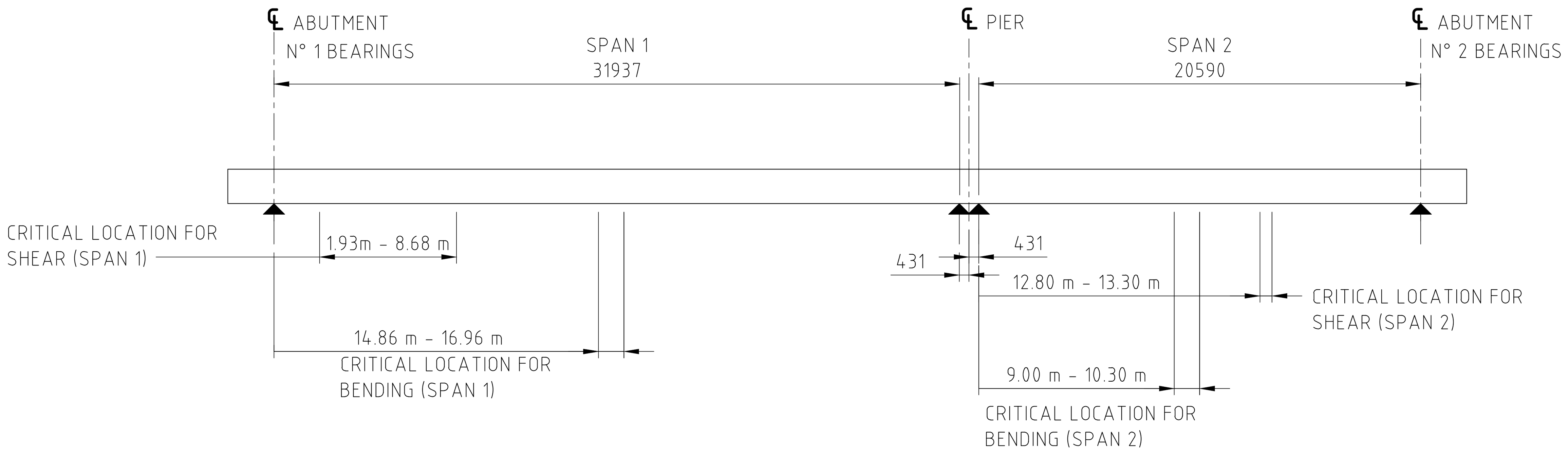
All structures must be designed such that elements, other than barriers, that are constructed initially do not require modification or strengthening for the ultimate configuration.

Design shall ensure any planned bridge widenings will be economic and practical.

## 1 - SPAN CONFIGURATION

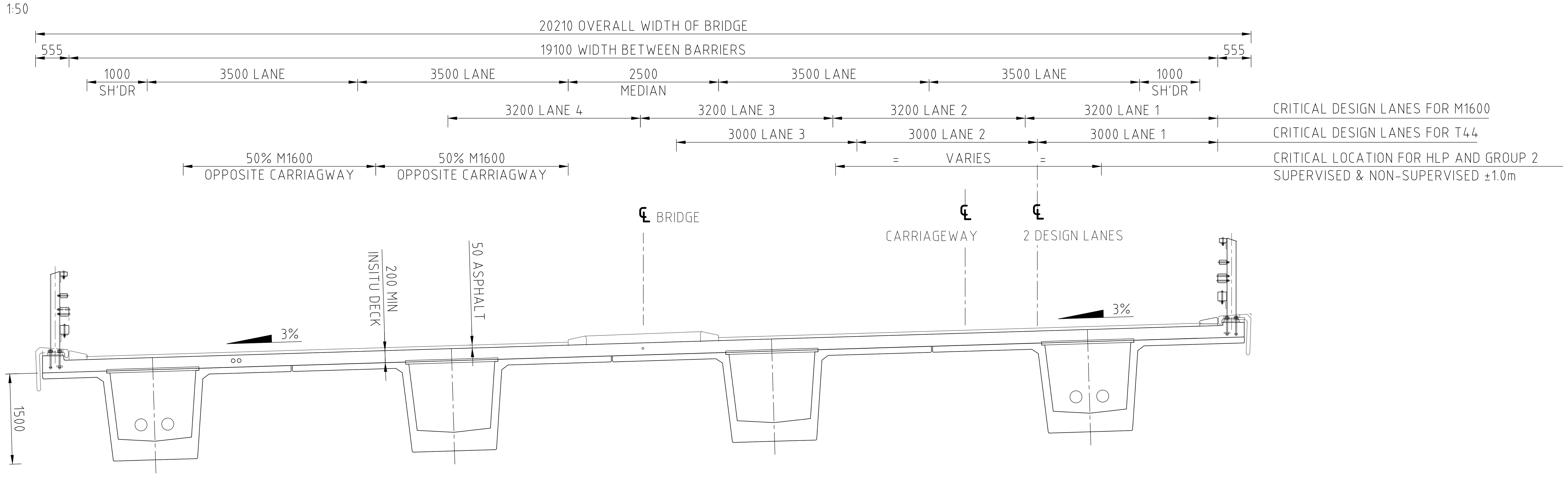
1:200

(SIMPLY SUPPORTED SPANS WITH LINK SLABS OVER PIER)



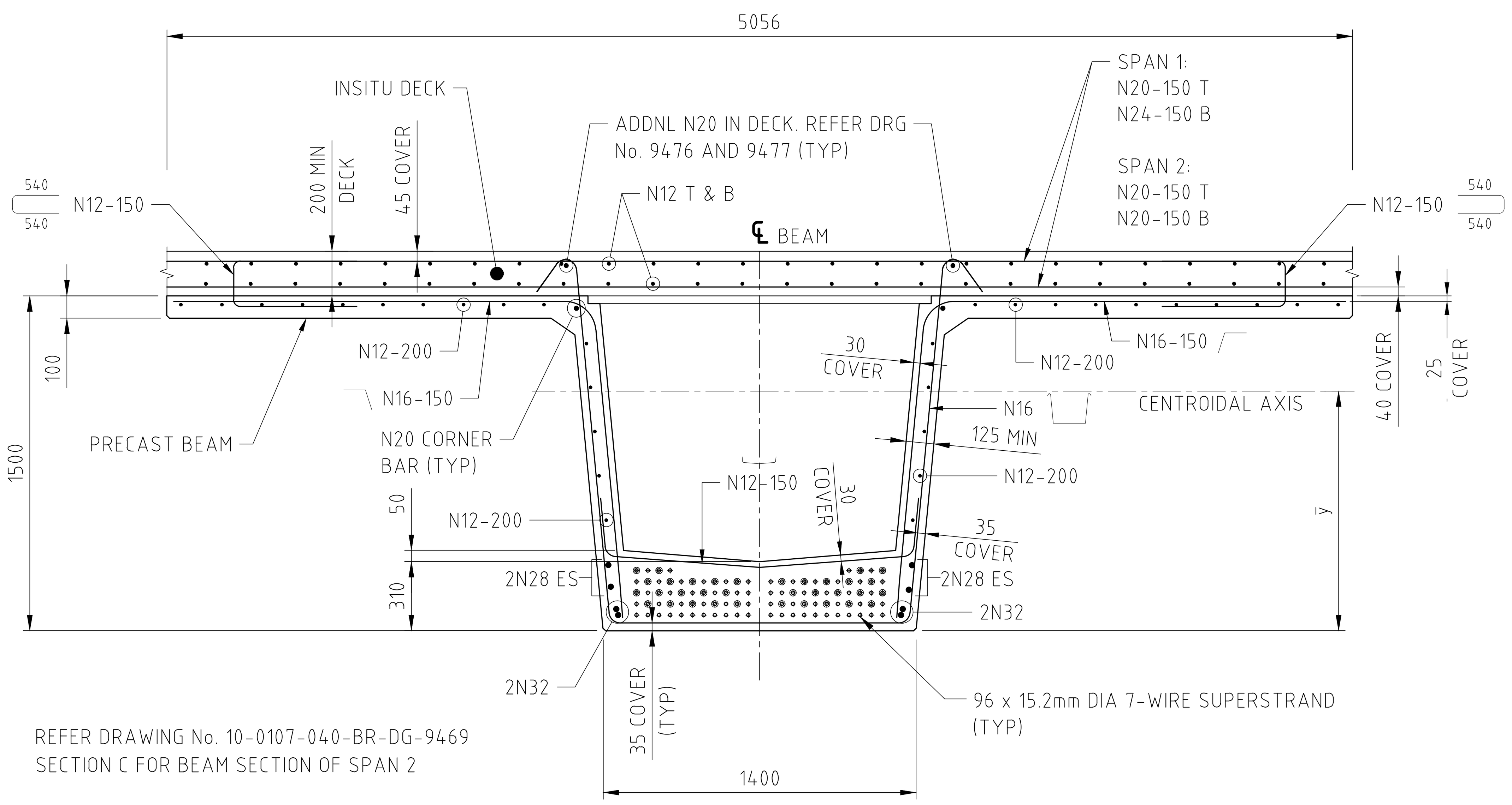

## 2 - TYPICAL DESIGN CROSS SECTION

1:50




## 3 - TYPICAL COMPOSITE BEAM SECTION AND PROPERTIES

1:20




REFER DRAWING No. 10-0107-040-BR-DG-9469
SECTION C FOR BEAM SECTION OF SPAN 2

## TABLE 1 - CHARACTERISTICS OF THE COMPOSITE SECTION

| | BENDING | | SHEAR | |
|---|---|---|---|---|
| CRITICAL SPAN & BEAM | SPAN 1 BEAM 4 | SPAN 2 BEAM 8 | SPAN 1 BEAM 4 | SPAN 2 BEAM 8 |
| DISTANCE FROM A1 END SUPPORT (m) | 14.86 TO 16.96 | 9.00 TO 10.30 | 1.93 | 12.80 TO 13.30 |
| No. OF TENDONS | 96 | 40 | 62 | 40 |
| $A_{cp}$ ($m^2$) | 2.656 | 2.656 | 2.656 | 2.656 |
| $A_{ct}$ ($m^2$) | 0.609 | 0.520 | 0.687 | 0.520 |
| $A_g$ ($m^2$) | 2.102 MIN | 1.907 MIN | 2.102 MIN | 1.907 MIN |
| $A_o$ ($m^2$) | 2.084 | 2.112 | 1.976 | 2.112 |
| $A_{oh}$ ($m^2$) | 2.363 | 2.363 | 2.354 | 2.363 |
| $A_{pt}$ ($mm^2$) [$\frac{1}{2}$ DEPTH] | 13728 | 5720 | 8306* | 5720 |
| $A_{sc}$ ($mm^2$) [ABOVE $d_n$] | 6341 | 3125 | 6341 | 3125 |
| $A_{st}$ ($mm^2$) [$\frac{1}{2}$ DEPTH] | 6132 | 3164 | 6584 | 3164 |
| $A_{sv}$ ($mm^2$) | 402 | 226 | 628 | 402 |
| $b_{ef}$ (m) | 5.056 | 5.056 | 5.056 | 5.056 |
| $b_v$ (mm) | 250 | 300 | 400 | 300 |
| d (m) | 1.543 | 1.592 | 1.547 | 1.592 |
| $d_n$ (mm) at $\phi M_U$ (FROM TOP) | 203.6 | 116.6 | 152.1 | 116.6 |
| $d_o$ (m) | 1.630 | 1.630 | 1.630 | 1.630 |
| $d_p$ (m) | 1.545 | 1.605 | 1.559 | 1.605 |
| $d_{sc}$ (m) | 0.098 | 0.072 | 0.098 | 0.072 |
| $d_v$ (m) | 1.439 | 1.433 | 1.392 | 1.433 |
| $I_g$ ($m^4$) | 0.7944 MIN | 0.6842 MIN | 0.7944 MIN | 0.6842 MIN |
| $M_S$ (kNm) | 24172 | 11078 | N/A | N/A |
| $P_v$ (kN) | 0 | 0 | 0 | 0 |
| s (mm) | 150 | 150 | 150 | 150 |
| $t_w$ (mm) | 125 | 150 | 200 | 150 |
| $u_c$ (m) | 6.540 | 6.540 | 6.540 | 6.540 |
| $u_h$ (m) | 6.169 | 6.169 | 6.157 | 6.169 |
| z (m) | 1.439 | 1.356 | 1.331 | 1.356 |
| $\alpha_v$ (°) | 90 | 90 | 90 | 90 |
| $\bar{y}$ (m) | 1.159 | 1.165 | N/A | N/A |
| $\sigma_{cp}$ (MPa) | 6.2 | 3.0 | 3.7 | 3.0 |
| $\sigma_{pu}$ (MPa) | 1652 | 1652 | 1652 | 1652 |
| $\phi M_U$ (kNm) | 30492 | 14094 | 20927 | 14094 |
| $0.25\phi T_{cr}$ (kNm) | 406 | 391 | 517 | 391 |

## MATERIAL PROPERTIES

### PRESTRESSING:

$f_{pb}$ = 1830 MPa
$f_{py}$ = 1501 MPa
$E_p$ = 195,000 MPa
$A_{pt}$ = 143$mm^2$ PER TENDON
JACKING STRESS = 1208 MPa

### CONCRETE:

$f'_c$ = 50 MPa (PRECAST BEAM)
$f'_c$ = 40 MPa (INSITU SLAB)
$E_c$ = 34800 MPa (PRECAST BEAM)
$E_c$ = 32800 MPa (INSITU SLAB)

### REINFORCEMENT:

$f_{sy}$ = 500 MPa
$E_s$ = 200,000 MPa

## DSS NOTES:

1. ALL LONGITUDINAL REINFORCEMENT AND PRESTRESSING ARE INCLUDED IN THE DETERMINATION OF $\phi M_U$. ACTUAL BENDING CAPACITY FOR EACH VEHICLE SHALL BE ADJUSTED BY THE RATIO OF THE LONGITUDINAL TENSILE FORCE DEMAND FROM BENDING ACTION OVER TOTAL TENSILE FORCE DEMAND INCLUDING SHEAR AND TORSION (SEE TABLES 2 AND 3).
2. $M_s$ BASED ON LIMITING THE INCREMENT IN STEEL STRESS NEAR THE TENSION FACE TO 200MPa.
3. EXTERNAL BEAMS VARY IN FLANGE WIDTH. THE CAPACITY GIVEN IS FOR THE CROSS SECTION WITH THE MINIMUM FLANGE WIDTH.
4. $\phi M_U$ AND $\sigma_{pu}$ HAS BEEN CALCULATED BASED ON AS 5100.5:2017 CLAUSE 8.1.7.
5. $A_{sv}$ AT THE CRITICAL SECTION FOR SHEAR HAS IGNORED THE D-SHIFT OF SHEAR REINFORCEMENT IN ACCORDANCE WITH AS 5100.5:2017 CLAUSE 8.3.2.3.
6. DEVELOPMENT ZONES OF DE-BONDED STRANDS WERE CHECKED FOR ANY CONCRETE TENSION FROM SLS LOADING ($2L_p$ CHECK) IN ACCORDANCE WITH AS 5100.5:2017 CLAUSE 13.3.2.2.
7. *DENOTES EFFECTIVE $A_{pt}$ DERIVED THROUGH LINEAR INTERPOLATION WITHIN A PRESTRESS DEVELOPMENT ZONE.

SCALE 1:200 1:20 1:50

| Rev | DATE | DESCRIPTION | APPROVED |
|---|---|---|---|

METADATA
GROUND SURVEY STANDARD:
DATE OF CAPTURE:
MAPPING SURVEY STANDARD:
DATE OF CAPTURE:
MAIN ROADS PROJECT ZONE:
HEIGHT DATUM:

mainroads WESTERN AUSTRALIA
INFRASTRUCTURE DELIVERY DIRECTORATE

DESIGNED | VERIFIER
DRAWN | DESIGN MANAGER APPROVED
CHECKED | ALLIANCE PROJECT MANAGER APPROVED
DRAWING PURPOSE

PROJECT TITLE
DRAWING TITLE
SIMPLY SUPPORTED BRIDGE
DESIGN SUMMARY - SHEET N° 1 OF 3
SAMPLE ONLY
SHEET A1
MRWA DRAWING No.
LOCAL AUTHORITY | MAIN ROADS RESPONSIBILITY AREA
STATUS | DRAWING NUMBER | REV

## TABLE 2 – DESIGN ACTIONS FOR ULS MOMENT AT CRITICAL LOCATIONS – LONGITUDINAL EFFECTS

| VEHICLE LOAD CASE | CRITICAL SPAN & BEAM | CRITICAL LOCATION FROM A1 END SUPPORT (m) | PERMANENT ACTIONS | | | ACCOMPANYING ACTIONS | | | LIVE LOAD ACTIONS | | | % LONG REINF. DEMAND FROM BENDING (REFER NOTE 9) | CONTROLLING DESIGN CASE | LOAD FACTORS FOR RATING VEHICLE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $M_{PE,ULS}$ (kNm) | $V_{PE}$ (kN) | $T_{PE}$ (kNm) | $M_{CE,ULS}$ (kNm) | $V_{CE}$ (kN) | $T_{CE}$ (kNm) | $M_{LL,ULS}$ (kNm) | $V_{LL}$ (kN) | $T_{LL}$ (kNm) | | | |
| M1600 (4 LANES) | SPAN 1 BEAM 4 | 16.96 | 11659 | 92 | 239 | 0 | 0 | 0 | 18320 | 9 | 319 | 97% | M2 | $\gamma_Q$=1.8, DLA=0.3 |
| T44 (2 LANES) | SPAN 2 BEAM 8 | 10.30 | 5223 | 22 | 51 | 0 | 0 | 0 | 5656 | 135 | -51 | 100% | M1 | $\gamma_Q$=2.0, DLA=0.4 |
| HLP 320 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 14.96 | 11677 | 93 | 334 | 1685 | 0 | -29 | 15285 | 156 | 378 | 96% | M2 | $\gamma_Q$=1.5, DLA=0.1 |
| HLP 400 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 16.16 | 11717 | 105 | 379 | 1650 | -38 | -62 | 18085 | 133 | 347 | 96% | M2 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 1, VEHICLE 1 (2 LANES) | SPAN 2 BEAM 8 | 9.10 | 5165 | 107 | 47 | 0 | 0 | 0 | 3130 | -116 | -49 | 100% | M1 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 2 (3 LANES) | SPAN 1 BEAM 4 | 14.96 | 11677 | 93 | 334 | 0 | 0 | 0 | 8464 | 103 | 172 | 96% | M2 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 3 (3 LANES) | SPAN 1 BEAM 4 | 14.96 | 11677 | 93 | 334 | 0 | 0 | 0 | 9357 | 108 | 190 | 96% | M2 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 4 (3 LANES) | SPAN 1 BEAM 4 | 14.96 | 11677 | 93 | 334 | 0 | 0 | 0 | 9086 | -100 | 186 | 96% | M2 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 2, VEHICLE 1 – 3.01* | SPAN 2 BEAM 8 | 9.00 | 5155 | 116 | 47 | 0 | 0 | 0 | 3286 | -86 | -110 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 1 – 3.70* | SPAN 2 BEAM 8 | 9.00 | 5155 | 116 | 47 | 0 | 0 | 0 | 3290 | -91 | -87 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 2 – 3.01* | SPAN 2 BEAM 8 | 9.10 | 5165 | 107 | 47 | 0 | 0 | 0 | 3720 | 111 | -65 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 2 – 3.70* | SPAN 2 BEAM 8 | 9.10 | 5165 | 107 | 47 | 0 | 0 | 0 | 3575 | 103 | -56 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 4* | SPAN 2 BEAM 8 | 10.30 | 5223 | 22 | 51 | 0 | 0 | 0 | 5096 | 69 | -73 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 4 – NS* | SPAN 2 BEAM 8 | 10.30 | 5223 | 22 | 51 | 0 | 0 | 0 | 6022 | 82 | -86 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.3 |
| GROUP 2, VEHICLE 5* | SPAN 2 BEAM 8 | 10.30 | 5223 | 22 | 51 | 0 | 0 | 0 | 6686 | 80 | -138 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 5 – NS* | SPAN 2 BEAM 8 | 10.30 | 5223 | 22 | 51 | 0 | 0 | 0 | 7901 | 94 | -163 | 100% | M1 | $\gamma_Q$=1.5, DLA=0.3 |
| GROUP 2, VEHICLE 7 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 16.16 | 11717 | 105 | 379 | 199 | -7 | -15 | 19749 | 109 | 370 | 96% | M2 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 8 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 16.16 | 11717 | 105 | 379 | 199 | -7 | -15 | 24834 | 147 | 455 | 96% | M2 | $\gamma_Q$=1.5, DLA=0.1 |

### TABLE 2 NOTES:

1. *DENOTES NO ADVERSE ACTIONS FROM CO-EXISTING LOADS IN THE OPPOSITE CARRIAGEWAY. ACTIONS FROM ACCOMPANYING LOADS ARE CONSIDERED SEPARATELY FOR BENDING, SHEAR AND TORSION.
2. *DENOTES CRITICAL LOCATIONS AT $2L_P$ DEVELOPMENT LENGTH OF DE-BONDED STRANDS IN ACCORDANCE WITH AS 5100.5:2017 CL.13.3.2.2. (NOT APPLICABLE FOR THIS EXAMPLE).
3. CRITICAL LOCATION IS FROM THE CENTRELINE OF BEARING AT LOWEST CHAINAGE OF THE RESPECTIVE SPAN.
4. DESIGN ACTIONS HAVE BEEN FACTORED BY ACCOMPANYING LANE FACTOR / MULTIPLE LANE MODIFICATION FACTOR, LOAD FACTOR AND DYNAMIC LOAD ALLOWANCE AS APPROPRIATE.
5. TORSION IS CONSIDERED WHEN TOTAL FACTORED TORSION EXCEEDS THE TORSION LIMIT AS PER AS 5100.5:2017 CL.8.2.1.2.
6. NEGATIVE $T_{CE}$ OR $T_{LL}$ INDICATE OPPOSITE DIRECTION TO $T_{PE}$.
7. ACCOMPANYING DESIGN ACTIONS ARE THE MOST ADVERSE CONTRIBUTIONS FROM 50% M1600 LOADING IN THE OPPOSITE CARRIAGEWAY. THESE LOADS ARE DEDUCTED FROM THE AVAILABLE CAPACITY, TOGETHER WITH PERMANENT DESIGN ACTIONS.
8. THE LONGITUDINAL BENDING LOAD RATING HAS BEEN CARRIED OUT AT VARIOUS SECTIONS ALONG THE SPAN AND AT EACH SECTION THE CASE WITH MAXIMUM TOTAL MOMENT ($M^*_{MAX}$), CORRESPONDING SHEAR AND TORSION HAS BEEN EXAMINED.
9. % LONG. REINF. DEMAND FROM BENDING REPRESENTS THE LONGITUDINAL TENSILE FORCE DEMAND ASSOCIATED WITH THE DESIGN BENDING MOMENT AS A PROPORTION OF THE TOTAL TENSILE FORCE DEMAND ON THE CROSS-SECTION ACCOUNTING FOR THE EFFECTS OF AXIAL FORCE, BENDING MOMENT, SHEAR FORCE AND TORSIONAL MOMENT.
10. FOR DEFINITION OF CONTROLLING DESIGN CASE REFER TO MRWA BBDIM (DOC 3912/02/04) SECTION 4.

## TABLE 3 – DESIGN ACTIONS FOR COMBINED SHEAR AND TORSION AT CRITICAL LOCATIONS – LONGITUDINAL EFFECTS

| VEHICLE LOAD CASE | CRITICAL SPAN & BEAM | CRITICAL LOCATION FROM A1 END SUPPORT (m) | PERMANENT ACTIONS | | | ACCOMPANYING ACTIONS | | | LIVE LOAD ACTIONS | | | CAPACITY | | | | CONTROLLING DESIGN CASE | LOAD FACTORS FOR RATING VEHICLE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $M_{PE,ULS}$ (kNm) | $V_{PE}$ (kN) | $T_{PE}$ (kNm) | $M_{CE,ULS}$ (kNm) | $V_{CE}$ (kN) | $T_{CE}$ (kNm) | $M_{LL,ULS}$ (kNm) | $V_{LL}$ (kN) | $T_{LL}$ (kNm) | $\phi V_{US,VT}$ (kN) | % TRANS REINF. DEMAND FROM SHEAR (REFER NOTE 2) | $\phi V_{UC}$ (kN) | $\phi V_{U,MAX}$ (kN) | | |
| M1600 (4 LANES) | SPAN 1 BEAM 4 | 1.93 | 1933 | 1370 | 378 | 0 | 0 | 0 | 3318 | 2331 | 480 | N/A | N/A | 1184 | 6447 | V2 | $\gamma_Q$=1.8, DLA=0.3 |
| T44 (2 LANES) | SPAN 2 BEAM 8 | 12.80 | 4896 | 246 | 71 | 0 | 0 | 0 | 4705 | 615 | -68 | N/A | N/A | 398 | 5505 | V3 | $\gamma_Q$=2.0, DLA=0.4 |
| HLP 320 (1 LANE 50% M1600) | SPAN 2 BEAM 8 | 1.93 | 1422 | 904 | 178 | 58 | 36 | -71 | 2074 | 1321 | 84 | N/A | N/A | 938 | 3476 | V1 | $\gamma_Q$=1.5, DLA=0.1 |
| HLP 400 (2 LANES 50% M1600) | SPAN 1 BEAM 4 | 2.19 | 2757 | 1318 | 383 | 787 | 138 | 0 | 3701 | 2089 | 1005 | N/A | N/A | 1146 | 6468 | V2 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 1, VEHICLE 1 (2 LANES) | SPAN 2 BEAM 8 | 12.80 | 4896 | 246 | 71 | 0 | 0 | 0 | 2514 | 326 | -49 | N/A | N/A | 397 | 5506 | V3 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 2 (2 LANES) | SPAN 1 BEAM 4 | 8.68 | 9241 | 683 | 348 | 0 | 0 | 0 | 5915 | 726 | 251 | N/A | N/A | 623 | 4256 | V2 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 3 (2 LANES) | SPAN 1 BEAM 4 | 8.68 | 9241 | 683 | 348 | 0 | 0 | 0 | 6791 | 847 | 325 | 1707 | 71% | 630 | 4251 | V5 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 1, VEHICLE 4 (2 LANES) | SPAN 1 BEAM 4 | 8.68 | 9241 | 683 | 348 | 0 | 0 | 0 | 6368 | 784 | 310 | N/A | N/A | 668 | 4224 | V2 | $\gamma_Q$=2.0, DLA=0.4 |
| GROUP 2, VEHICLE 1 – 3.01* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 2409 | 356 | 190 | 847 | 73% | 446 | 5420 | V5 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 1 – 3.70 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 8.68 | 9241 | 683 | 348 | 200 | 6 | 0 | 4536 | 572 | 133 | 1830 | 78% | 589 | 4283 | V5 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 2 – 3.01* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 2848 | 420 | 216 | 855 | 74% | 443 | 5425 | V5 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 2 – 3.70* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 2753 | 402 | 194 | 867 | 75% | 436 | 5437 | V5 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 4* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 3424 | 494 | -200 | N/A | N/A | 404 | 5494 | V3 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 4 – NS* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 4046 | 583 | -236 | N/A | N/A | 404 | 5494 | V3 | $\gamma_Q$=1.5, DLA=0.3 |
| GROUP 2, VEHICLE 5* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 4106 | 602 | 333 | 837 | 72% | 447 | 5419 | V5 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 5 – NS* | SPAN 2 BEAM 8 | 13.30 | 4748 | 293 | -76 | 0 | 0 | 0 | 4853 | 711 | 394 | 837 | 72% | 447 | 5419 | V5 | $\gamma_Q$=1.5, DLA=0.3 |
| GROUP 2, VEHICLE 7 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 1.93 | 1933 | 1370 | 378 | 151 | 11 | 59 | 3119 | 2357 | 497 | N/A | N/A | 1185 | 6447 | V2 | $\gamma_Q$=1.5, DLA=0.1 |
| GROUP 2, VEHICLE 8 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 1.93 | 1933 | 1370 | 378 | 151 | 11 | 59 | 4030 | 2930 | 559 | N/A | N/A | 1184 | 6447 | V2 | $\gamma_Q$=1.5, DLA=0.1 |

### TABLE 3 NOTES:

IN ADDITION TO TABLE 2 NOTES:

1. THE LOAD RATING HAS BEEN CARRIED OUT ALONG THE SPAN AND AT EACH SECTION THE FOLLOWING THREE CASES WERE EXAMINED:
   - MAXIMUM TOTAL MOMENT ($M^*_{MAX}$), CORRESPONDING SHEAR AND TORSION.
   - MAXIMUM TOTAL SHEAR ($V^*_{MAX}$), CORRESPONDING MOMENT AND TORSION.
   - MAXIMUM TOTAL TORSION ($T^*_{MAX}$), CORRESPONDING MOMENT AND SHEAR.
2. % TRANSVERSE REINFORCEMENT DEMAND FROM SHEAR REPRESENTS THE TRANSVERSE SHEAR REINFORCEMENT DEMAND ASSOCIATED WITH THE SHEAR, $V^*$ AS A PROPORTION OF THE TOTAL DEMAND ASSOCIATED WITH COMBINED SHEAR, $V^*$ AND TORSION, $T^*$ (SCENARIO V5 ONLY).
3. FOR DEFINITION OF CONTROLLING DESIGN CASE REFER TO MRWA BBDIM (DOC 3912/02/04) SECTION 4.

SCALE

Rev. | DATE | DESCRIPTION | APPROVED

METADATA
GROUND SURVEY STANDARD:
DATE OF CAPTURE:
MAPPING SURVEY STANDARD:
DATE OF CAPTURE:
MAIN ROADS PROJECT ZONE:
HEIGHT DATUM:

mainroads WESTERN AUSTRALIA

DESIGNED | VERIFIER
DRAWN | DESIGN MANAGER APPROVED
CHECKED | ALLIANCE PROJECT MANAGER APPROVED
DRAWING PURPOSE

PROJECT TITLE
DRAWING TITLE: SIMPLY SUPPORTED BRIDGE DESIGN SUMMARY – SHEET N° 2 OF 3 SAMPLE ONLY
SHEET
MRWA DRAWING No.
LOCAL AUTHORITY | MAIN ROADS RESPONSIBILITY AREA
STATUS | DRAWING NUMBER | REV

## TABLE 4 - SLS DESIGN MOMENTS AT CRITICAL LOCATIONS - LONGITUDINAL EFFECTS

| VEHICLE LOAD CASE | CRITICAL SPAN & BEAM | CRITICAL LOCATION FROM A1 END SUPPORT (m) | $M_{PE,SLS}$ (kNm) | $M_{CE,SLS}$ (kNm) | $M_{LL,SLS}$ (kNm) | LOAD FACTORS FOR RATING VEHICLE |
|---|---|---|---|---|---|---|
| M1600 (3 LANES) | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 4780 | $\gamma_Q$=1.0, DLA=0.3 |
| T44 (2 LANES) | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 2828 | $\gamma_Q$=1.0, DLA=0.4 |
| HLP 320 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 16.26 | 9424 | 915 | 10282 | $\gamma_Q$=1.0, DLA=0.1 |
| HLP 400 (1 LANE 50% M1600) | SPAN 1 BEAM 4 | 16.26 | 9424 | 915 | 12060 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 1, VEHICLE 1 (2 LANES) | SPAN 2 BEAM 8 | 9.10 | 4143 | 0 | 1565 | $\gamma_Q$=1.0, DLA=0.4 |
| GROUP 1, VEHICLE 2 (2 LANES) | SPAN 2 BEAM 8 | 9.00 | 4135 | 0 | 2098 | $\gamma_Q$=1.0, DLA=0.4 |
| GROUP 1, VEHICLE 3 (2 LANES) | SPAN 2 BEAM 8 | 9.00 | 4135 | 0 | 2344 | $\gamma_Q$=1.0, DLA=0.4 |
| GROUP 1, VEHICLE 4 (2 LANES) | SPAN 2 BEAM 8 | 9.00 | 4135 | 0 | 2186 | $\gamma_Q$=1.0, DLA=0.4 |
| GROUP 2, VEHICLE 1 - 3.01* | SPAN 2 BEAM 8 | 9.00 | 4135 | 0 | 2191 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 1 - 3.70* | SPAN 2 BEAM 8 | 9.00 | 4135 | 0 | 2194 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 2 - 3.01* | SPAN 2 BEAM 8 | 9.10 | 4143 | 0 | 2480 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 2 - 3.70* | SPAN 2 BEAM 8 | 9.10 | 4143 | 0 | 2383 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 4* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 3397 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 4 - NS* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 4015 | $\gamma_Q$=1.0, DLA=0.3 |
| GROUP 2, VEHICLE 5* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 4457 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 5 - NS* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 5267 | $\gamma_Q$=1.0, DLA=0.3 |
| GROUP 2, VEHICLE 7* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 6344 | $\gamma_Q$=1.0, DLA=0.1 |
| GROUP 2, VEHICLE 8* | SPAN 2 BEAM 8 | 10.30 | 4190 | 0 | 7802 | $\gamma_Q$=1.0, DLA=0.1 |

### TABLE 4 NOTES:

1. DESIGN MOMENT HAS BEEN FACTORED BY MULTIPLE LANE MODIFICATION FACTOR / ACCOMPANYING LANE FACTOR, LOAD FACTOR AND DYNAMIC LOAD ALLOWANCE AS APPROPRIATE.
2. *DENOTES NO ADVERSE ACTIONS FROM ACCOMPANYING LOADS IN THE OPPOSITE CARRIAGEWAY.
3. CRITICAL LOCATION IS FROM THE CENTRELINE OF BEARING AT LOWEST CHAINAGE OF THE SPAN.
4. THE LONGITUDINAL BENDING RATING IS FOR THE CRITICAL LOCATION WHICH HAS BEEN IDENTIFIED THROUGH CHECKING AT VARIOUS SECTIONS ALONG THE SPAN.

## TABLE 5 - LOAD RATINGS AT CRITICAL LOCATIONS

| VEHICLE LOAD CASE | VEHICLE WIDTH (m) | SAG MOMENT LOAD RATING (%) | | SHEAR LOAD RATING (%) | CONTROLLING DESIGN CASE |
|---|---|---|---|---|---|
| | | SERVICEABILITY LIMIT STATE | ULTIMATE LIMIT STATE | ULTIMATE LIMIT STATE | |
| M1600 | 2.4 | 144% | 98% | 108% | M2 |
| T44 | 2.2 | 244% | 157% | 162% | M1 |
| HLP 320 | 3.6 | 135% | 104% | 105% | M2 |
| HLP 400 | 4.5 | 115% | 89% | 101% | M2 |
| GROUP 1, VEHICLE 1 | 2.4 | 443% | 285% | 304% | M1 |
| GROUP 1, VEHICLE 2 | 2.4 | 331% | 208% | 226% | M2 |
| GROUP 1, VEHICLE 3 | 2.4 | 296% | 188% | 194% | M2 |
| GROUP 1, VEHICLE 4 | 2.4 | 318% | 194% | 201% | M2 |
| GROUP 2, VEHICLE 1 - 3.01 | 3.01 | 317% | 272% | 280% | M1 |
| GROUP 2, VEHICLE 1 - 3.70 | 3.70 | 317% | 272% | 302% | M1 |
| GROUP 2, VEHICLE 2 - 3.01 | 3.01 | 280% | 240% | 239% | V5 |
| GROUP 2, VEHICLE 2 - 3.70 | 3.70 | 291% | 250% | 251% | M1 |
| GROUP 2, VEHICLE 4 | 3.70 | 203% | 174% | 219% | M1 |
| GROUP 2, VEHICLE 4 - NS | 3.70 | 172% | 147% | 185% | M1 |
| GROUP 2, VEHICLE 5 | 3.01 | 155% | 133% | 164% | M1 |
| GROUP 2, VEHICLE 5 - NS | 3.01 | 131% | 112% | 139% | M1 |
| GROUP 2, VEHICLE 7 | 4.81 | 109% | 88% | 105% | M2 |
| GROUP 2, VEHICLE 8 | 6.22 | 88% | 70% | 85% | M2 |

### TABLE 5 NOTES:

1. TRANSVERSE DESIGN ACTIONS HAVE BEEN EXAMINED AND ARE GREATER THAN THE RATINGS FOR LONGITUDINAL BENDING AND SHEAR.
2. SUBSTRUCTURE RATINGS HAVE BEEN EXAMINED AND ARE GREATER THAN THE RATINGS FOR LONGITUDINAL BENDING AND SHEAR.

## GENERAL NOTES:

1. PERMANENT EFFECTS COMPRISE OF SELF WEIGHT AND SUPERIMPOSED DEAD LOADS.
2. NOMINAL ALLOWANCES FOR THE BRIDGE PER METRE RUN OF DECK ARE:
   - 100mm ASPHALT BETWEEN KERBS - 2.2kPa
   - BARRIER - 1kN/m
   - SELF WEIGHT BEAMS - 26kN/m$^3$
   - SELF WEIGHT INSITU DECK, MINIMUM THICKNESS - 5.1 kPa
   - (PLUS ADDITIONAL SDL ALLOWANCES FOR THICKNESS OF UP TO 50mm AT ABUTMENT ENDS DUE TO ROAD PROFILE AND BEAM HOG)
3. DESIGN VEHICLE IS SM1600 AS PER AS 5100.2:2017.
4. LOAD EFFECTS HAVE BEEN BASED ON AN ELASTIC ANALYSIS OF THE STRUCTURE. THE STRUCTURE IS SIMPLY SUPPORTED.
5. DYNAMIC LOAD ALLOWANCE (DLA) IS 0.3 FOR SM1600 IN ACCORDANCE WITH AS 5100.2-2017. DLA IS 0.4 FOR T44 AND GROUP 1 VEHICLES, AND 0.1 FOR HLP AND GROUP 2 VEHICLES IN ACCORDANCE WITH AS 5100.7:2017 AND MRWA BBDIM (DOC 3912/02/04).
6. DLA IS 0.3 FOR UNSUPERVISED GROUP 2 VEHICLES. FOR DETAILS OF UNSUPERVISED MOVEMENT, REFER TO SECTION 4.3.4 OF MRWA BBDIM (DOC 3912/02/04).
7. HLP AND GROUP 2 VEHICLE POSITIONING IS IN ACCORDANCE WITH MRWA BBDIM (DOC 3912/02/04).
8. SHEAR LOAD RATINGS INCLUDE AN ALLOWANCE FOR COMBINED SHEAR AND TORSION. COMBINED SHEAR AND TORSION IS CALCULATED IN ACCORDANCE WITH AS 5100.5-2017 SECTION 8.2 AND MAIN ROADS WA SES CIRCULAR 01/20.
9. DISTRIBUTION FACTORS ARE THE RATIO BETWEEN MAXIMUM DESIGN ACTION FOR A BEAM AND TOTAL DESIGN ACTION FOR ALL BEAMS, FOR PURE BENDING AND SHEAR (WITHOUT THE EFFECTS TORSION). DISTRIBUTION FACTORS ARE FOR THE CRITICAL LOCATION FOR EACH VEHICLE, DISTRIBUTED PERPENDICULAR TO THE BEAM. CO-EXISTING EFFECTS ARE NOT INCLUDED.

## TABLE 6 - DISTRIBUTION FACTORS

| VEHICLE LOAD CASE | SAG MOMENT AT CRITICAL LOCATION | | SHEAR AT CRITICAL LOCATION | |
|---|---|---|---|---|
| | CRITICAL SPAN & BEAM | DF | CRITICAL SPAN & BEAM | DF |
| M1600 | SPAN 1 BEAM 4 | 0.467 | SPAN 1 BEAM 4 | 0.524 |
| T44 | SPAN 2 BEAM 8 | 0.674 | SPAN 2 BEAM 8 | 0.597 |
| HLP 320 | SPAN 1 BEAM 4 | 0.603 | SPAN 2 BEAM 8 | 0.546 |
| HLP 400 | SPAN 1 BEAM 4 | 0.605 | SPAN 1 BEAM 4 | 0.633 |
| GROUP 1, VEHICLE 1 | SPAN 2 BEAM 8 | 0.686 | SPAN 2 BEAM 8 | 0.565 |
| GROUP 1, VEHICLE 2 | SPAN 1 BEAM 4 | 0.464 | SPAN 1 BEAM 4 | 0.694 |
| GROUP 1, VEHICLE 3 | SPAN 1 BEAM 4 | 0.472 | SPAN 1 BEAM 4 | 0.734 |
| GROUP 1, VEHICLE 4 | SPAN 1 BEAM 4 | 0.472 | SPAN 1 BEAM 4 | 0.674 |
| GROUP 2, VEHICLE 1 - 3.01 | SPAN 2 BEAM 8 | 0.673 | SPAN 2 BEAM 8 | 0.630 |
| GROUP 2, VEHICLE 1 - 3.70 | SPAN 2 BEAM 8 | 0.672 | SPAN 1 BEAM 4 | 0.697 |
| GROUP 2, VEHICLE 2 - 3.01 | SPAN 2 BEAM 8 | 0.676 | SPAN 2 BEAM 8 | 0.539 |
| GROUP 2, VEHICLE 2 - 3.70 | SPAN 2 BEAM 8 | 0.651 | SPAN 2 BEAM 8 | 0.503 |
| GROUP 2, VEHICLE 4 | SPAN 2 BEAM 8 | 0.669 | SPAN 2 BEAM 8 | 0.565 |
| GROUP 2, VEHICLE 4 - NS | SPAN 2 BEAM 8 | 0.669 | SPAN 2 BEAM 8 | 0.565 |
| GROUP 2, VEHICLE 5 | SPAN 2 BEAM 8 | 0.663 | SPAN 2 BEAM 8 | 0.576 |
| GROUP 2, VEHICLE 5 - NS | SPAN 2 BEAM 8 | 0.663 | SPAN 2 BEAM 8 | 0.576 |
| GROUP 2, VEHICLE 7 | SPAN 1 BEAM 4 | 0.596 | SPAN 1 BEAM 4 | 0.629 |
| GROUP 2, VEHICLE 8 | SPAN 1 BEAM 4 | 0.582 | SPAN 1 BEAM 4 | 0.615 |

## NOTATION:

| | | |
|---|---|---|
| $A_{cp}$ | = | TOTAL AREA ENCLOSED BY OUTSIDE PERIMETER OF CONCRETE SECTION ENCASING TORSION REINFORCEMENT |
| $A_{ct}$ | = | AREA OF CONCRETE ON FLEXURAL TENSION SIDE OF MEMBER |
| $A_{st}$ | = | CROSS-SECTIONAL AREA OF LONGITUDINAL TENSILE REINFORCEMENT IN THE TENSILE HALF-DEPTH OF THE SECTION |
| $b_v$ | = | TOTAL WIDTH OF WEB(S) FOR SHEAR; OR EFFECTIVE WIDTH OF THE CRITICAL WEB IN AS 5100.5:2017 CL.8.2.1.2 ONLY. |
| CE | = | CO-EXISTING EFFECTS |
| DF | = | DISTRIBUTION FACTOR |
| DL | = | DEAD LOADS ARE PERMANENT LOADS UNLIKELY TO VARY DURING THE USE OF THE STRUCTURE |
| DLA | = | DYNAMIC LOAD ALLOWANCE |
| $d_n$ | = | DEPTH OF NEUTRAL AXIS FROM EXTREME COMPRESSION FIBRE |
| $I_g$ | = | SECOND MOMENT OF AREA OF THE GROSS CROSS-SECTION |
| LL | = | LIVE LOAD |
| $M_{CE,SLS}$ | = | SERVICEABILITY DESIGN MOMENT DUE TO ACCOMPANYING LIVE LOAD |
| $M_{CE,ULS}$ | = | ULTIMATE DESIGN MOMENT DUE TO ACCOMPANYING LIVE LOAD |
| $M_{LL,SLS}$ | = | SERVICEABILITY DESIGN MOMENT DUE TO LIVE LOAD |
| $M_{LL,ULS}$ | = | ULTIMATE DESIGN MOMENT DUE TO LIVE LOAD |
| $M_{PE,SLS}$ | = | SERVICEABILITY DESIGN MOMENT DUE TO ALL PERMANENT EFFECTS |
| $M_{PE,ULS}$ | = | ULTIMATE DESIGN MOMENT DUE TO ALL PERMANENT EFFECTS |
| $M_S$ | = | SERVICEABILITY CAPACITY IN BENDING (PER BEAM) |
| NS | = | NON-SUPERVISED VEHICLE MOVEMENT. REFER MRWA BBDIM (DOC 3912/02/04). |
| PE | = | ALL PERMANENT EFFECTS (DL + SDL) |
| s | = | CENTRE-TO-CENTRE SPACING OF SHEAR/TORSIONAL REINFORCEMENT |
| SDL | = | SUPERIMPOSED DEAD LOADS ARE PERMANENT LOADS WHICH MAY VARY DURING THE USE OF THE STRUCTURE (IE. TEMPORARY KERBS, BARRIERS, FOOTPATHS, ASPHALT THICKNESS) |
| SLS | = | SERVICEABILITY LIMIT STATE |
| $T_{CE}$ | = | ULTIMATE DESIGN TORSION DUE TO CO-EXISTING LIVE LOAD |
| $T_{LL}$ | = | ULTIMATE DESIGN TORSION DUE TO LIVE LOAD |
| $T_{PE}$ | = | ULTIMATE DESIGN TORSION DUE TO ALL PERMANENT EFFECTS |
| $u_c$ | = | LENGTH OF OUTSIDE PERIMETER OF CONCRETE CROSS-SECTION ENCASING TORSION REINFORCEMENT |
| ULS | = | ULTIMATE LIMIT STATE |
| $V_{CE}$ | = | ULTIMATE DESIGN SHEAR DUE TO ACCOMPANYING LIVE LOAD |
| $V_{LL}$ | = | ULTIMATE DESIGN SHEAR DUE TO LIVE LOAD |
| $V_{PE}$ | = | ULTIMATE DESIGN SHEAR DUE TO ALL PERMANENT EFFECTS |
| z | = | INTERNAL MOMENT LEVER ARM OF THE SECTION |
| $\bar{y}$ | = | DISTANCE TO NEUTRAL AXIS FROM BOTTOM OF SECTION |
| $\gamma_{DL}$ | = | DEAD LOAD FACTOR. REFER AS 5100.2:2017 T.6.2. |
| $\gamma_Q$ | = | TRAFFIC LOAD FACTOR. REFER AS 5100.2:2017 T.7.10(A) & MRWA BBDIM (DOC 3912/02/04). |
| $\gamma_{SDL}$ | = | SUPERIMPOSED DEAD LOAD FACTOR. REFER AS 5100.2:2017 T.6.3. |
| $\sigma_{cp}$ | | AVERAGE INTENSITY OF EFFECTIVE PRESTRESS IN CONCRETE AT THE CENTROID, OR AT THE JUNCTION OF THE WEB AND FLANGE WHEN THE CENTROID LIES INSIDE THE FLANGE. REFER AS 5100.5:2017 CL.8.2.1.2.. |
| $\phi$ | = | CAPACITY REDUCTION FACTOR (0.8 FOR BENDING AND 0.7 FOR SHEAR/TORSION) |
| $\phi M_U$ | = | ULTIMATE CAPACITY IN BENDING (PER BEAM) |
| $\phi V_{US,VT}$ | = | CONTRIBUTION OF SHEAR REINFORCEMENT TO THE ULTIMATE SHEAR STRENGTH (PER BEAM) ADJUSTED BY TRANSVERSE SHEAR REINFORCEMENT DEMAND ASSOCIATED WITH THE SHEAR AS A PROPORTION OF THE TOTAL DEMAND FROM COMBINED SHEAR AND TORSION, ONLY WHEN SCENARIO V5 IS CONTROLLING. |

FOR ALL OTHER DEFINITIONS REFER TO AS 5100.5:2017.

SCALE

| Rev | DATE | DESCRIPTION | APPROVED |
|---|---|---|---|

**METADATA**
GROUND SURVEY STANDARD:
DATE OF CAPTURE:
MAPPING SURVEY STANDARD:
DATE OF CAPTURE:
MAIN ROADS PROJECT ZONE:
HEIGHT DATUM:

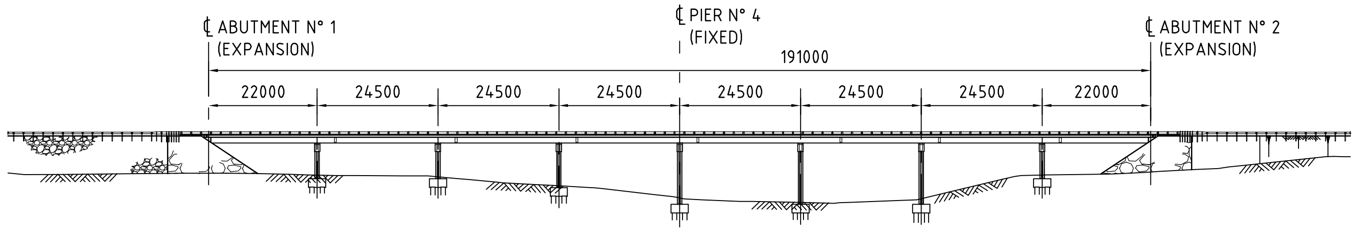
mainroads WESTERN AUSTRALIA
INFRASTRUCTURE DELIVERY DIRECTORATE

DESIGNED | VERIFIER
DRAWN | DESIGN MANAGER APPROVED
CHECKED | ALLIANCE PROJECT MANAGER APPROVED
DRAWING PURPOSE

PROJECT TITLE
DRAWING TITLE
SIMPLY SUPPORTED BRIDGE
DESIGN SUMMARY - SHEET N° 3 OF 3
SAMPLE ONLY

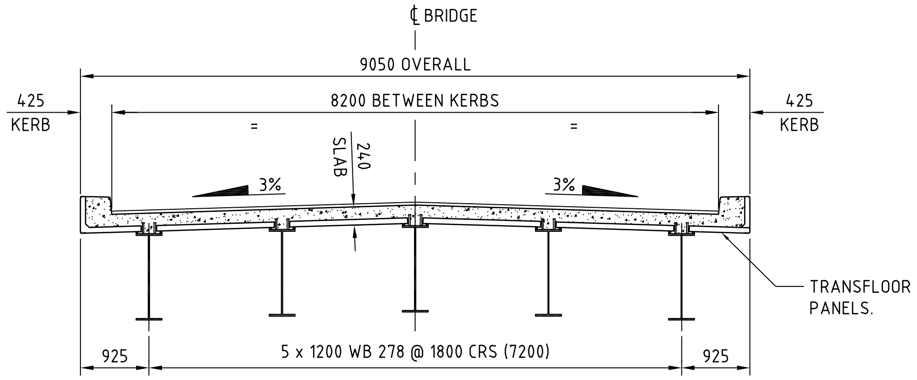
SHEET A1
MRWA DRAWING No.
LOCAL AUTHORITY | MAIN ROADS RESPONSIBILITY AREA
STATUS | DRAWING NUMBER | REV

## 1 SPAN CONFIGURATION




ELEVATION
1:750

## 2 DESIGN CRITICAL SECTION

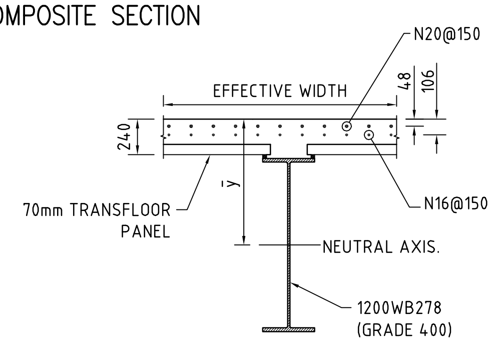



TYPICAL CROSS SECTION
1:50

## 3 PROPERTIES OF TRANSFORMED COMPOSITE SECTION

TABLE 3.1

| LOCATION | ABUTMENTS & PIERS | MID SPAN |
|---|---|---|
| EFFECTIVE WIDTH (m) | 1.8 | 1.8 |
| A ($m^2$) | 0.04152 | 0.08895 |
| I comp. ($m^4$) | 0.01058 | 0.01941 |
| $\bar{y}$ (m) | 0.714 | 0.380 |




fsy = 500MPa
f'c = 40MPa
Ec = 35000MPa
Es = 200000MPa

NOTES (TABLE 3.1)

1. I comp. IS BASED ON AN EQUIVALENT STEEL SECTION.

## 4 TRANSVERSE DISTRIBUTION FACTORS FOR LIVE LOADS

TABLE 4.1

| VEHICLE | SAG | HOG | SHEAR |
|---|---|---|---|
| M1600 (2 VEH.) | 0.271 | 0.263 | 0.267 |
| T44 & GROUP 1 VEHICLES (2 VEH.) | 0.288 | 0.281 | 0.290 |
| L44 (2 LANES) | - | 0.251 | - |
| GROUP 2 VEHICLE 4 (3.7m O/A WIDTH) | 0.267 | 0.292 | 0.301 |
| GROUP 2 VEHICLE 5 (3.01m O/A WIDTH) | 0.269 | 0.298 | 0.315 |
| GROUP 2 VEHICLE 7 (4.81m O/A WIDTH) | 0.275 | 0.278 | 0.279 |
| GROUP 2 VEHICLE 8 (6.22m O/A WIDTH) | 0.232 | 0.243 | 0.242 |

NOTES (TABLE 4.1)

1. M1600 AND T44/L44 DISTRIBUTION FACTORS ARE FOR THE MOST CRITICAL LANE LOADING, INCLUDING THE APPROPRIATE ACCOMPANYING LANE FACTORS OR LANE MODIFICATION FACTORS.
2. DISTRIBUTION FACTORS DO NOT INCLUDE DYNAMIC LOAD ALLOWANCE.
3. GROUP 2 VEHICLE DISTRIBUTION FACTORS ARE GIVEN FOR A SINGLE VEHICLE TRAVELLING DOWN THE CENTRE LINE OF THE BRIDGE ± 1 METRE.

THIS DRAWING IS AN AMENDMENT OF THE APPROVED DRAWING. ________________

## 5 TYPICAL DESIGN MOMENTS AND SHEARS AT CRITICAL LOCATIONS

TABLE 5.1 ULTIMATE LIMIT STATE

| LOAD CASE | SAG (kNm) | HOG (kNm) | SHEAR (kN) | NOTES |
|---|---|---|---|---|
| PE | 295 | 1223 | 104 | 1.4SDL+1.2S+1.5DS |
| M1600 (2 VEH.) | 3212 | 2647 | 900 | $\gamma_L$ =1.8, DLA=0.3 |
| T44 (2 VEH.) | 2245 | - | 552 | $\gamma_L$ =2.0, DLA=0.4 |
| L44 (2 LANES) | - | 1453 | - | $\gamma_L$ =2.0, DLA=0.4 |
| GROUP 2 VEHICLE 4 | 1922 | 1352 | 560 | $\gamma_L$ =1.5, DLA=0.1 |
| GROUP 2 VEHICLE 5 | 2587 | 1537 | 759 | $\gamma_L$ =1.5, DLA=0.1 |
| GROUP 2 VEHICLE 7 | 3844 | 2576 | 1027 | $\gamma_L$ =1.5, DLA=0.1 |
| GROUP 2 VEHICLE 8 | 4323 | 2968 | 1186 | $\gamma_L$ =1.5, DLA=0.1 |

TABLE 5.2 SERVICEABILITY LIMIT STATE

| LOAD CASE | HOG (kNm) | NOTES |
|---|---|---|
| PE | 967 | 1.0SDL+1.0S+1.0DS |
| M1600 (2 VEH.) | 1471 | $\gamma_L$ =1.0, DLA=0.3 |

NOTES (TABLE 5.1 & 5.2)

1. LOAD EFFECTS HAVE BEEN CALCULATED BASED ON AN ELASTIC ANALYSIS OF THE STRUCTURE. CRACKED SECTION PROPERTIES ARE ADOPTED OVER 15% OF SPAN LENGTH BOTH SIDES OF PIERS.

## 6 AVAILABLE LIVE LOAD CAPACITY

TABLE 6.1 MOMENT CAPACITY

| ULTIMATE LIMIT STATE | SAG (kNm) | HOG (kNm) |
|---|---|---|
| $\phi M_u - M_{ULT.PE} - 1.0\,M_{DT}$ | 6686 | 2673 |

| SERVICEABILITY LIMIT STATE | HOG (kNm) |
|---|---|
| $M_s$ (fs = 280MPa) - $M_{SERV.PE} - 0.5M_{DT} - 0.5M_{VW}$ | 3416 |

TABLE 6.2 SHEAR CAPACITY

| ULTIMATE LIMIT STATE | SHEAR (kN) |
|---|---|
| $\phi V_u - V_{ULT.PE} - 1.0\,V_{DT}$ | 2921 |

NOTES (TABLE 6.1 & 6.2)

1. DEAD LOAD EFFECTS PRODUCED FROM CONSTRUCTION STAGING HAVE BEEN INCORPORATED IN A REDUCED STEEL BEAM fy USED TO CALCULATE ULTIMATE MOMENT CAPACITY ABOVE.
2. SERVICEABILITY CAPACITY IN BENDING IS BASED ON A MAX STRESS OF 280MPa IN THE DECK STEEL REINFORCEMENT.

## SYMBOLS USED

- A - EFFECTIVE AREA OF SECTION
- DLA - DYNAMIC LOAD ALLOWANCE
- DS - DIFFERENTIAL SETTLEMENT
- Ec - MODULUS OF ELASTICITY OF CONCRETE AT 28 DAYS
- Es - MODULUS OF ELASTICITY OF STEEL
- f'c - CHARACTERISTIC COMPRESSIVE CYLINDER STRENGTH OF CONCRETE AT 28 DAYS
- fs - ALLOWABLE STRESS IN ORDINARY REINFORCEMENT
- fsy - YIELD STRESS OF ORDINARY REINFORCEMENT
- I comp. - EFFECTIVE SECOND MOMENT OF AREA OF COMPOSITE SECTION
- $M_{DT}$ - SERVICEABILITY MOMENT DUE TO DIFFERENTIAL TEMPERATURE
- $M_S$ - SERVICEABILITY CAPACITY IN BENDING
- $M_{SERV.PE}$ - SERVICEABILITY MOMENT DUE TO ALL PERMANENT EFFECTS
- $M_U$ - ULTIMATE CAPACITY IN BENDING
- $M_{ULT.PE}$ - ULTIMATE MOMENT DUE TO ALL PERMANENT EFFECTS
- $M_{VW}$ - SERVICEABILITY MOMENT DUE TO VERTICAL WIND
- PE - PERMANENT EFFECTS
- S - SHRINKAGE
- SDL - SUPERIMPOSED DEAD LOAD
- $V_{DT}$ - SERVICEABILITY SHEAR DUE TO DIFFERENTIAL TEMPERATURE
- $V_{ULT.PE}$ - ULTIMATE SHEAR DUE TO ALL PERMANENT EFFECTS
- $V_V$ - ULTIMATE CAPACITY IN SHEAR
- $\bar{y}$ - DISTANCE TO NEUTRAL AXIS, AS PER SECTION 3
- $\gamma_L$ - LOAD FACTOR FOR LIVE LOAD
- $\phi$ - CAPACITY REDUCTION FACTOR

NOTES:

1. FOR GENERAL NOTES REFER TO SHEET 2

| No. | DATE | DESCRIPTION | AUTHORISED |
|---|---|---|---|
| 1 | 21/03/12 | UPDATED TITLEBLOCK. | |

AMENDMENTS

CLIENT

CONSULTANT

**PLANNING AND TECHNICAL SERVICES DIRECTORATE**
**STRUCTURES ENGINEERING**
Telephone (08) 9323 4111 Fax (08) 9323 4136

| | |
|---|---|
| FILE No. | JOB No. |
| DRAWN | DESIGNED |
| CHECKED | VERIFIED |
| APPROVED | |

A1

SCALE 1:750 1:50 MICROFILM DATE

Government of Western Australia
mainroads WESTERN AUSTRALIA

GUIDE DRAWING
CONTINUOUS COMPOSITE BRIDGE
DESIGN SUMMARY - SHEET N° 1 OF 2
SAMPLE ONLY

LOCAL AUTHORITY ( )

DRAWING NUMBER AMEND. 1

## 7 LOAD RATING

TABLE 7.1

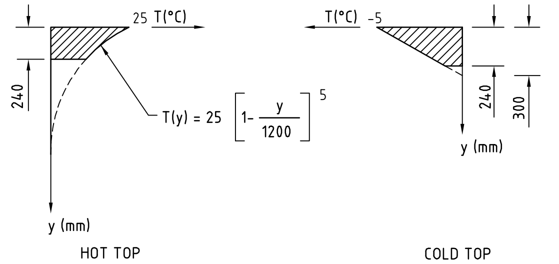
| DESIGN SUMMARY SHEET AND LOAD RATING DESIGNATION | VEHICLE TYPE AND CONFIGURATION | TRANSVERSE VEHICLE CONFIGURATION (EXCLUDING PRIME MOVER) | SAG MOMENTS LOAD RATING % ULTIMATE LIMIT STATE | HOG MOMENTS LOAD RATING % SERVICEABILITY LIMIT STATE | HOG MOMENTS LOAD RATING % ULTIMATE LIMIT STATE | SHEAR LOAD RATING % ULTIMATE LIMIT STATE |
|---|---|---|---|---|---|---|
| M1600 | 360kN 360kN 6kN/m 360kN 360kN; 3.75 VARIES 6.25 MIN 5.0 | 2.0 | 208% | 232% | 101% | 325% |
| T44 / L44 | 48kN 96kN 96kN 96kN 96kN; 3.7 1.2 VARIES 3 TO 8 1.2 | 1.8 | 298% | 470%* | 184%* | 529% |
| GROUP ONE - VEHICLE ONE | 6t 18t 18t; 4.9 1.2 7.25 1.2; PRIME MOVER + 18 TONNE TANDEM (G.C.M. = 42 TONNES) | 1.8 | 403% | 886% | 347% | 647% |
| GROUP ONE - VEHICLE TWO | 6t 18t 18t 27t; 4.9 1.2 3 1.2 6.5 1.2 1.2; PRIME MOVER + 18 TONNE DOLLY + 27 TONNE TRIAXLE (G.C.M. = 69 TONNES) | 1.8 | 287% | 529% | 207% | 441% |
| GROUP ONE - VEHICLE THREE | 6t 18t 18t 36t; 4.9 1.2 3 1.2 6.5 1.2 1.2 1.2; PRIME MOVER + 18 TONNE DOLLY + 36 TONNE STANDARD QUAD (G.C.M. = 78 TONNES) | 1.8 | 272% | 460% | 180% | 393% |
| GROUP ONE - VEHICLE FOUR | 6t 18t 18t 36t; 4.9 1.2 3 1.2 6.5 1.2 2.4 1.2; PRIME MOVER + 18 TONNE DOLLY + 36 TONNE SPREAD QUAD (G.C.M. = 78 TONNES) | 1.8 | 290% | 457% | 179% | 418% |
| GROUP TWO - VEHICLE ONE | 6t 18t 36t 54t; 4.9 1.2 3 1.2 6.5 1.8 1.8; PRIME MOVER + 36 TONNE DOLLY + 54 TONNE TRIAXLE (G.C.M. = 114 TONNES) | 0.81 & 1.5; 0.85 0.85; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 3.01m & 3.7m | 567% | 654% | 332% | 725% |
| GROUP TWO - VEHICLE TWO | 6t 18t 36t 72t; 4.9 1.2 3 1.2 6.5 1.2 2.4 1.2; PRIME MOVER + 36 TONNE DOLLY + 72 TONNE SPREAD QUAD (G.C.M. = 132 TONNES) | 0.81 & 1.5; 0.85 0.85; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 3.01m & 3.7m | 476% | 546% | 285% | 628% |
| GROUP TWO - VEHICLE FOUR | 6t 18t 36t 144t; 4.9 1.2 3 1.2 6.5 1.8 1.8 1.8 1.8 1.8 1.8 1.8; PRIME MOVER + 36 TONNE DOLLY + 144 TONNE 8-LINE PLATFORM (G.C.M. = 204 TONNES) | 1.5; 0.85 0.85; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 3.7m | 348% | 379% | 198% | 522% |
| | | | 202%** | 232%** | 121%** | 318%** |
| GROUP TWO - VEHICLE FIVE | 6t 18t 216t; 4.9 1.2 5.5 1.45 …; PRIME MOVER + 216 TONNE 12-LINE PLATFORM (G.C.M. = 240 TONNES) | 0.85 0.81 0.85; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 3.01m | 258% | 333% | 174% | 385% |
| | | | 137%** | 188%** | 98%** | 273%** |
| GROUP TWO - VEHICLE SEVEN | 6t 18t 432t; 4.9 1.2 5.0 1.55 …; PRIME MOVER + 432 TONNE 16-LINE PLATFORM 3 FILE (G.C.M. = 456 TONNES) | 0.73 1.09 0.73 1.09 0.73; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 4.81m | 174% | 199% | 104% | 284% |
| GROUP TWO - VEHICLE EIGHT | 6t 18t 576t; 4.9 1.2 5.0 1.55 …; PRIME MOVER + 576 TONNE 16-LINE PLATFORM 4 FILE (G.C.M. = 600 TONNES) | 0.73 1.09 0.73 0.68 0.73 1.09 0.73; FOR OVERALL SPREAD TO OUTSIDE OF OUTER WHEELS OF 6.22m | 155% | 173% | 90% | 246% |

NOTES: (TABLE 7.1)

1. GROUP ONE VEHICLES ARE TREATED AS PER T44 LOADINGS (E.G. DYNAMIC LOAD ALLOWANCE, LIMIT STATE LOAD FACTORS, LANE REDUCTION ETC.) ALL OF THE BRIDGE DESIGN LANES ARE ASSUMED OCCUPIED BY THE SAME VEHICLE TYPE.
2. GROUP TWO VEHICLES ARE TREATED AS PER HLP LOADING WITH RESPECT TO DYNAMIC LOAD ALLOWANCE, LIMIT STATE LOAD FACTORS, POSITIONING DOWN CENTRELINE OF BRIDGE ± 1 METRE.
3. '*' REPRESENTS L44 CONTROLLING.
4. '**' REPRESENTS UNSUPERVISED MOVEMENT. REFER TO SECTION 4.3.4 OF BRIDGE BRANCH DESIGN INFORMATION MANUAL (DOC 3912/02/04) FOR DETAILS. GROUP 2 VEHICLES 4 & 5 ARE PLACED AT A CLEARANCE OF 200mm BETWEEN THE KERB AND TYRES FOR UNSUPERVISED MOVEMENT CONDITION. DLA = 0.3

## GENERAL NOTES:

1. PERMANENT EFFECTS COMPRISE THE SUM OF SUPERIMPOSED DEAD LOAD, SHRINKAGE AND AN ALLOWANCE FOR 5mm DIFFERENTIAL SETTLEMENT AT ABUTMENTS AND PIERS.
2. LOAD EFFECTS DUE TO DIFFERENTIAL SETTLEMENT ARE BASED ON AN EFFECTIVE MODULUS OF ELASTICITY THAT TAKES INTO ACCOUNT LONG TERM EFFECTS DUE TO CREEP FOR 40MPa CONCRETE.
3. NOMINAL ALLOWANCES FOR THE BRIDGE DECK ARE:
   75mm BITUMINOUS CONCRETE SURFACING BETWEEN KERBS - 13.53 kN/m
   GUARDRAILS - 2 kN/m (TOTAL)
   STEEL BEAMS - 13.6 kN/m (5 N°s)
   SELF WEIGHT OF DECK INCLUDING KERBS = 62 kN/m
4. THE TEMPERATURE EFFECT IS CAUSED BY THE THERMAL GRADIENTS AS SHOWN:-




5. THE DESIGN VEHICLE IS SM1600 AS PER AS 5100. THE WORST DESIGN EFFECT IS PRODUCED BY 2 LANES OF M1600 LOADING.

| No. | DATE | DESCRIPTION | AUTHORISED |
|---|---|---|---|
| 2 | 21/03/12 | ADDED UNSUPERVISED MOVEMENT FOR GROUP 2 - VEHICLE 4 & 5. UPDATED TITLEBLOCK. | |
| 1 | 04/05/06 | AMENDED GROUP ONE - VEHICLE FOUR'S VEHICLE CONFIGURATION TONNAGE. | |

AMENDMENTS

CLIENT

CONSULTANT

PLANNING AND TECHNICAL SERVICES DIRECTORATE
STRUCTURES ENGINEERING

Telephone (08) 9323 4111 Fax (08) 9323 4136

FILE No. JOB No.
DRAWN DESIGNED
CHECKED VERIFIED
APPROVED

SCALE
MICROFILM DATE
A1

Government of Western Australia
mainroads WESTERN AUSTRALIA

GUIDE DRAWING
CONTINUOUS COMPOSITE BRIDGE
DESIGN SUMMARY - SHEET N° 2 OF 2
SAMPLE ONLY

LOCAL AUTHORITY ( )

DRAWING NUMBER AMEND. 2

THIS DRAWING IS AN AMENDMENT OF THE APPROVED DRAWING. ________________

# SECTION 3 – REFURBISHMENT AND STRENGTHENING DESIGN

This information is Section 3 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As the head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information

**A LIM**

SENIOR ENGINEER STRUCTURES

Date: 04/09/2023

**Document No: 3912/02-3**

**Controlled Copies shall be marked accordingly**

## SECTION 3

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 24/01/06 | Complete review for introduction of AS 5100-2004 and 6706-02-2227. |
| 3 | 2 | 30/07/10 | Addition of new section 3.4 |
| All | 3 | 30/08/18 | Complete review for introduction of AS5100-2017 |
| All | 4 | 04/09/23 | Amended 3.1 Stage 2 |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

Custodian Endorsement

M RAJAKARUNA

Structures Design & Standards Engineer

Date: 04/09/2023

# 3 REFURBISHMENT AND STRENGTHENING DESIGN

## 3.1 Introduction

This Section of the Design Information Manual shall be used for the design of refurbishment and strengthening of existing bridges.

The refurbishment and strengthening design process is essentially split into two basic stages as follows:

**Stage 1 – Load Rating**

This stage analyses the load capacity of the existing structure in its current condition to determine structural deficiencies. It involves modelling the bridge in its current form utilising all available information on actual geometric, material and condition parameters. The load rating stage shall be carried out in accordance with Section 4 “Load Rating Existing Bridges” for non-timber bridges, or Document No. 6706-02-2227 “Load Rating and Refurbishment Design Manual for Existing Timber Bridges” for timber bridges.

The designer is required to confirm the analysis vehicles with EBL.

**Stage 2 – Refurbishment and/or Strengthening Design**

This stage follows logically from stage 1, which has identified areas of structural deficiency or zones requiring strengthening to accommodate the required refurbishment (widening, raising, strengthening, additional traffic lanes etc). The refurbishment and/or strengthening design stage shall be carried out in accordance with the requirements detailed below.

Design shall be in accordance with the Code and Section 2 of this Manual except that design life may be modified as appropriate to the remaining service life of the existing bridge.

The designer is required to confirm the design vehicles with EBL.

## 3.2 Non-Timber Bridges

Refurbishment and strengthening designs shall be to the Code except where varied by the Bridge Branch Design Information Manual. If it has been demonstrated that Code standards cannot be achieved then a lower level of service may be accepted.

## 3.3 Timber Bridges

Refurbishment and strengthening designs for existing timber bridges shall be carried out in accordance with the “Load Rating and Refurbishment Design Manual for Existing Timber Bridges”, Document No. 6706-02-2227.

The design shall, as far as practical, include the appropriate standard repair details contained within the Pavements & Structures Engineering Practice Notes, Document Nos. 6702/02/221, 222, 223. Such details are provided for general information only. Although structural sizes are given, reference must be made to approved drawings for each specific job, and all standard details must be assessed and confirmed as suitable by an engineering analysis.

South-West Region and Structures Engineering are developing Project Standard Drawings. These are akin to Standard Drawings for timber maintenance/refurbishment. Until such time as these are authorised by SES, all Project Standard Drawings are to be treated as project specific drawings. That is, given a project specific drawing number and submitted for review alongside other project specific drawings.

## 3.4 Construction Considerations

During construction of a refurbishment or strengthening design it is common for the road to remain at least partially open to traffic. The Designer must provide for this in the design by either propping various elements as required or designating traffic restrictions that are required.

The bridge must be analysed for each stage during construction (for example, a two stage RCO construction), giving allowable traffic loads for at least the three modes of tri-axle group vehicles if applicable, outlined in Document No. 6706-02-2227 “Load Rating and Refurbishment Design Manual for Existing Timber Bridges” for timber bridges and the Group 1 Vehicle 2 and Group 2 Vehicle 1 at both spreads in as per Appendix D in Section 4 “Load Rating Existing Bridges” for non-timber bridges. This information is required for heavy load assessments during construction and shall be forwarded to EBL prior to construction.

Construction staging and associated load restrictions should also be specified on the Drawings in line with staged load assessments as above.

**SECTION 4 – LOAD RATING OF BRIDGES**

This information is Part 4 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

Engineer Bridge Loading is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As the head of Structures Engineering of Main Roads Western Australia,

I authorise this issue and the use of this Information.

**A LIM**
**SENIOR ENGINEER STRUCTURES**

Date: 01/10/2024

**Document No: 3912/02-4**

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**SECTION 4**

# CONTENTS

REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 24/01/06 | Complete review for introduction of AS 5100 (2004) and 6706-02-2227 |
| 6 | 2 | 10/01/07 | Section 4.3.4 - Paragraph 1 amended to include “..all design vehicles” |
| 21 | 3 | 14/3/07 | Note diagram amended |
| 8-10 | 4 | 04/03/11 | Differential temperature amended |
| All | 5 | 13/1/12 | Updated load rating vehicles and added Appendix F. |
| All | 6 | 06/07/22 | Complete review against AS 5100 (2017). |
| All | 7 | 04/09/23 | Minor update and corrections |
| All | 8 | 01/10/24 | Minor update and corrections |

Custodian Endorsement

P Rajasekera
Acting/Engineer Bridge Loading (EBL)
Date: 01/10/2024

# 4. LOAD RATING OF BRIDGES

## 4.1 INTRODUCTION

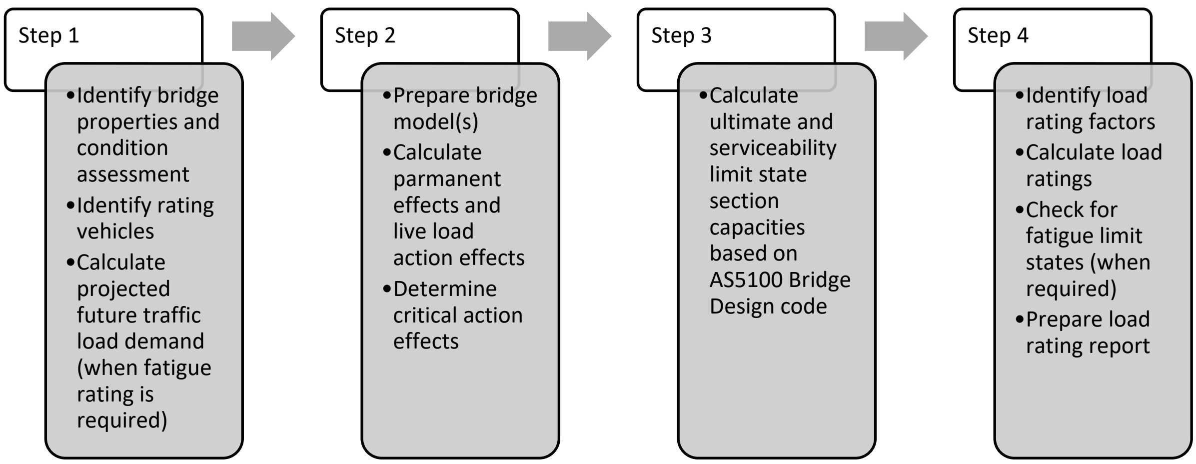
This Section of the Manual shall be used to determine the load rating of non-timber bridges in Western Australia (WA), and to determine the load limit posting requirements for bridges with deficient load capacity. Load rating of existing timber bridges shall be carried out in accordance with the “Load Rating and Refurbishment Design Manual for Existing Timber Bridges”, Document No. 6706-02-2227.

Load ratings are carried out on all new bridges as well as existing bridges. For new bridges the load rating results should be provided as part of the detailed drawing set and referred to as the Design Summary Sheets (DSS). If the load rating of an existing bridge indicates that a bridge is incapable of accommodating Vehicle Standard Regulation (VSR) loads, then, in the interests of public safety, the structure is either repaired or a load limit is posted.

The objective of load rating a bridge is to determine its safe live load carrying capacity in terms of the percentage of each rating vehicle’s action effect. These calculated bridge load ratings are used to update the bridge rating values in the MRWA bridge database, which are used for the assessment of heavy haulage movements throughout the State on publicly accessible roads.

All components of the bridge, including its foundation, should be considered to ensure that all relevant components are load rated. While the general analytical approach for load rating should follow the AS 5100 (2017) Bridge Design code, this Section 4 provides additional guidelines for a standardised approach for analysing bridges and reporting the load rating outcomes. Refer flowchart guideline in Figure 4.1 below.

## 4.2 FLOWCHART FOR NON-TIMBER BRIDGE LOAD RATING




Note: The above steps are guide only.

Figure 4.1 Bridge Load Rating Process Flowchart

## 4.3 IDENTIFY BRIDGE PROPERTIES AND CONDITION ASSESSMENT

The first step in carrying out a load rating is to obtain accurate up-to-date drawings of the bridge. These will usually be available from the MRWA Structures Engineering Drawing Office in pdf format. A full set of drawings should be obtained to be able to check for any non-standard features, with all as-constructed, repair, refurbishment details, etc noted.

Also obtain a copy of the latest detailed inspection report or arrange to have an inspection carried out if requested. Although assessments are usually on the basis of “as-new” condition, the actual state of the bridge is obviously important and could lead to a downgrading of the assessed capacity. Where actual material properties are known, these should be incorporated into the rating analysis.

It is also valuable to check the old bridge records and files in the Heavy Loads Group office. Although information on all bridges may not be available, these files often contain material not recorded elsewhere, which could have an important bearing on the bridge capacity, e.g. information on past load approvals, details of precast members, load tests, load limits, etc.

If detailed drawings are not available for a bridge, alternate methods such as local breakouts, destructive/non-destructive testing or bridge load testing can be utilised to determine the load rating within the provisions of the AS 5100 (2017) code. The details of these alternate methods are beyond the scope of this guideline.

Care is necessary when reading old drawings, especially pre-metric ones. Imperial dimensions can be converted readily, but care is needed with concrete strengths, reinforcement sizes and grades, prestressing steel grade, and structural steelwork sizes and grades.

### 4.3.1 Concrete

On old drawings, concrete strengths are given in pounds per square inch or “psi”, and on the older bridges this will be the crushing strength of a standard 6” cube, rather than the cylinder strength. The cube strength is not exactly the same as the cylinder strength. However, with concrete so old, and given that concrete crushing is rarely a limiting factor, it is usually accurate enough to make a direct conversion, as below:

3,000 psi = 20 MPa
4,500 psi = 30 MPa
6,000 psi = 40 MPa
7,500 psi = 50 MPa

Old concretes generally exceeded the minimum specification requirements for strength, so using concrete strengths on the drawings will generally be conservative. If the decision to strengthen is based purely on the assumption of concrete strength, material sampling can be undertaken within the provisions of the AS 5100 (2017) code.

### 4.3.2 Reinforcement

Imperial reinforcement is called up either by diameter, e.g. ⅝” @ 6” c/c, or by a number, e.g. D9045 @ 8” c/c. The first is reasonably straightforward. In the second, the first digit, (or the first two if the bar diameter ≥ 1¼”), is the bar diameter in ⅛”, e.g., 9=1⅛”, the rest is the bar mark.

The grade of old reinforcement will generally be either of the following.

- ordinary mild steel (plain or deformed), with a yield of 230 MPa and an allowable stress of 125 MPa, also known as Grade S, or Structural Grade, or
- cold worked deformed, also known as CW60, with a yield of 410 MPa and an allowable stress of 170 MPa.
- Refer Section 6 for other reinforcement stress limits

The grade should be stated on the drawings, but generally only mild steel was available until the mid-late 60s, with cold worked steel gradually replacing it and taking over almost completely by the mid 70s. If in doubt refer to the bar schedules, which were usually included with the drawings. Mild steel could be bent round a 2D pin, whereas cold worked used a minimum of 4D. Take care though, as often cold worked was used for the main straight bars and mild steel for stirrups and ligatures. If still in doubt, it is possible to take a sample for testing, from a non-critical region, otherwise err on the side of caution. Refer also to DIS 3912/02-6 “Stress Limits in Structural Concrete” for allowable stresses for various steel grades.

A conversion table for imperial bars is provided below:

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Bar Diameter (inches) | ⅜” | ½” | ⅝” | ¾” | ⅞” | 1” | 1⅛” | 1¼” | 1⅜” |
| Bar Area (mm2) | 71 | 126 | 198 | 285 | 388 | 506 | 641 | 792 | 958 |

Refer also to the Steel Reinforcement Institute of Australia’s “Guide to Historical Steel Reinforcement in Australia” and AS 5100.7:2017 Appendix A.

### 4.3.3 Prestress

Prestressing steel details must be checked to obtain the correct properties. Usually, even on the older bridges, for wire or strand the yield will be around 1700 MPa and 1250 MPa for prestressing bar.

Sometimes, the tendon capacity can be reported in imperial tons per square inch. (1 ton = 2,240 lbs). For example, *0.276” dia. 100-110 Tons Ultimate H.T. Wire with stresses in wires after release to be 138,000psi at 85°F* can be interpreted as the ultimate strength (i.e., breaking strength) of the HT steel used in the 0.276” dia. prestressing wire was 100 to 110 tons per square inch. Taking a mean value of 105 tons/in², this equates to (105 x 2,240) = 235,200 psi. Usually, wires were pre-tensioned to about 70% UTS (ultimate tensile strength) and allowing for some losses, the stress in each wire at release would be about 60% of UTS – that is: (235,200 x 0.6) = 141,120psi which is close to the 138,000 psi quoted in the sketch.

One issue with older prestressing steel is that it will probably be normal relaxation, as low relaxation steel only became generally available in the mid-70s. This will considerably increase long-term losses. It may be difficult to identify the steel in older bridges, especially in prefabricated, precast elements, as a number of different types, including a lot of imported steel were used. With prestress it is not usually possible to take a sample for testing, except perhaps from the end of a pre-tensioned member, so again err on the side of caution.

**Structural Steelwork** - Structural steelwork in older bridges, e.g., rolled or fabricated beams, can be of a number of different sizes, grades and origins. For the more recent of the older bridges, i.e., post 1960, beams will probably be BHP RSJs or UBs, Grade 250. Section properties for some of these pre-metric beams are attached at Appendix A. Older beams may well be imported, usually from England, and detailed measurements will have to be taken to calculate section properties. Grade 250 can usually be assumed safely, but some really old beams, (e.g., Horseshoe and Barrack St Bridges), were made of a very brittle, high carbon steel and should be checked carefully, and perhaps a sample taken for testing. Higher strength grades should only be used where they are clearly indicated on the drawings.

## 4.4 IDENTIFY RATING VEHICLES

For non-timber bridges, unless otherwise specified, ratings shall be carried out for all design vehicles as per code and rating vehicles detailed in this section as well as any additional design vehicles specified for a particular bridge (ex. additional vehicles for Oversize and Over-Mass Vehicle Corridors). EBL shall also be consulted for any special vehicle loadings requiring load rating. Other loads needed to be considered for load rating shall be in accordance with AS 5100.7:2017 bridge assessment code. It is important to use the correct dimensions and axle loads. Where dimensions can vary, the full range of axle spacing must be investigated, as different lengths may be critical for sag and hog moments. Please note, for new bridge design, if there is any conflict between the specifications/requirements and guidelines presented in this document, EBL shall be consulted for resolution prior to proceeding for the final design.

### 4.4.1 SM1600 Vehicles

As per AS 5100 (2017) code. While positioning vehicles in the most onerous position, any temporary or mountable median/verge/footpath/kerb needs to be ignored and vehicles must be positioned between full width of the bridge between traffic barriers as per Cl. 7.5 of AS5100.2:2017.

### 4.4.2 T44 Trucks

The T44 truck load shall consist of the magnitude and positioning of loads described in Clause A2.2.2 of AS 5100.7:2004 code/Section 16 of Bridge Branch Design Information Manual (BBDIM). The dynamic load allowance (DLA) shall be calculated based on AS 5100.7:2017 Cl. 11.3.6 and lane modification factors (LMF) shall be calculated based on AS 5100.7:2004 Table A2. To identify the most critical loading pattern for multilane carriageways, all vehicle combinations shall be considered with appropriate (LMF) including the single vehicle case positioned in the most onerous position.

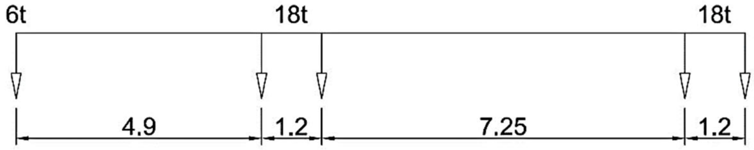
On longer bridges, there is a possibility of getting multiple loaded vehicles in a lane at the same time. Considering such ‘travel in convoy’ scenario, the minimum headway should be 15 metres measured between the rear axle of the front vehicle and the front axle of the rear vehicle in the same lane.

For any carriageway width, the vehicles shall be positioned in the most onerous position (while positioning vehicles in the most onerous position, any temporary or mountable median/verge/footpath/kerb needs to be ignored and vehicles must be positioned between traffic barriers) within the carriageway for the section under consideration but the centre-line of the dual tyre should not be closer than 600 mm to the face of the kerb or not closer than 1200 mm to the centre-line of the dual tyre of the accompanying T44 vehicle (refer to Figure A6 (a) of AS 5100.7:2004 code).

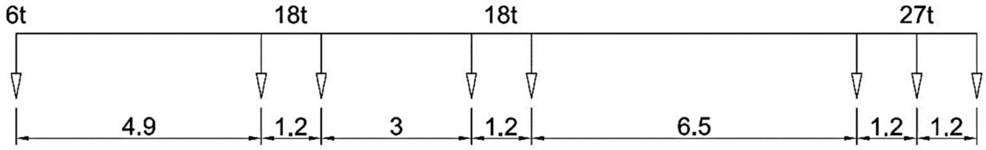
### 4.4.3 Group 1 Vehicles

Group 1 vehicles shall consist of the magnitude, spread and axle spacings of loads shown in Figure 4.2. The dynamic load allowance (DLA), lane modification factors (LMF) and vehicle position for the Group 1 Vehicles should be calculated as per T44 design vehicle above (consider T44 Vehicle replaced with Group 1 Vehicle while reading).

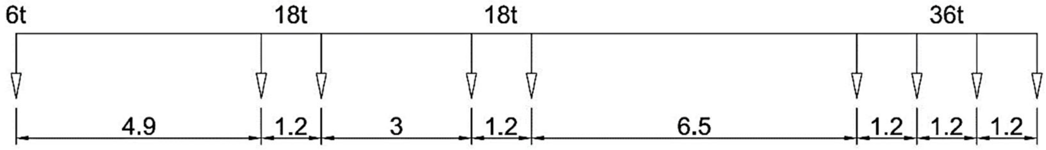
In absence of detailed analysis, the Group 1 Vehicle's distribution factors can be assumed to have the same value as a T44 design vehicle (not applicable with modified compression field theory (MCFT) analysis).



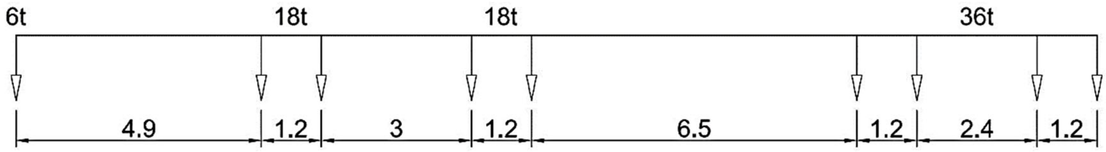

1. PRIME MOVER + 18 TONNE TANDEM
(G.C.M. = 42 TONNES)




2. PRIME MOVER + 18 TONNE DOLLY + 27 TONNE TRIAXLE
(G.C.M. = 69 TONNES)




3. PRIME MOVER + 18 TONNE DOLLY + 36 TONNE STANDARD QUAD
(G.C.M. = 78 TONNES)




4. PRIME MOVER + 18 TONNE DOLLY + 36 TONNE 484 QUAD
(G.C.M. = 78 TONNES)

NOTE
O/A WIDTH OF ALL VEHICLES 2.40m

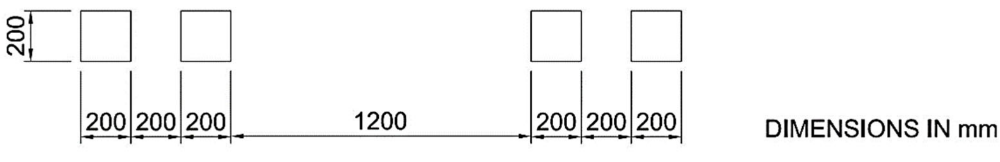


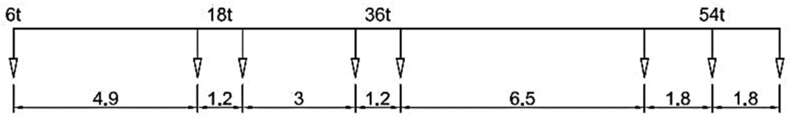

Figure 4.2 Magnitude, Spread and Axle Spacing of Loads for Group 1 Vehicles

### 4.4.4 HLP 320 and HLP 400

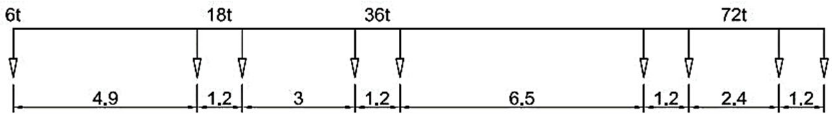
As per AS 5100 (2017) code except vehicle positioning. Vehicle positioning of HLP 320 and HLP 400 shall be in accordance with Fig 4.4. The positioning of HLP vehicles shall be the worst case of ignoring or considering temporary/ mountable median/verge/footpath/kerbs. Vehicles must be positioned within full width between barriers on the bridge to ensure the bridge is designed for road traffic to allow for future changes to lane markings.

### 4.4.5 Group 2 Vehicles (Supervised)

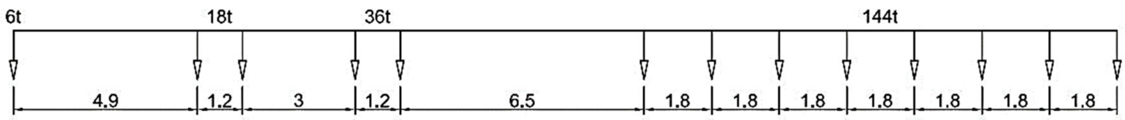
Group 2 Vehicles shall consist of the magnitude, spread and axle spacing of loads shown in Figure 4.3, with the Traffic load factor as per HLP vehicles. As the travel speed is controlled through appropriate supervision, the DLA shall be taken as 0.1. The positioning of Group 2 vehicles shall be, with reference to Figure 4.4, the worst case of ignoring or considering temporary/mountable median/verge/footpath/kerbs. Please note, after ignoring mountable median for divided carriageway bridges, the vehicle positioning should be as per Figure 4.4 (c) or (d) as appropriate instead of Figure 4.4 (a). Vehicles must be positioned within the full width between external barriers on the bridge to ensure the bridge is designed for road traffic to allow for future changes to lane markings.



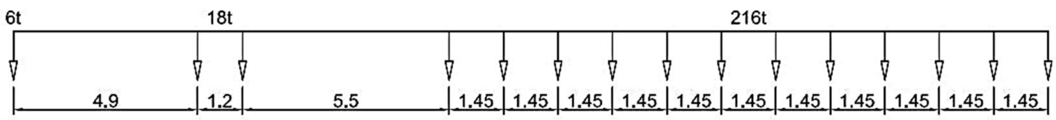

1. PRIME MOVER + 36 TONNE DOLLY + 54 TONNE TRIAXLE
   (G.C.M. = 114 TONNES)




2. PRIME MOVER + 36 TONNE DOLLY + 72 TONNE SPREAD QUAD
   (G.C.M. = 132 TONNES)

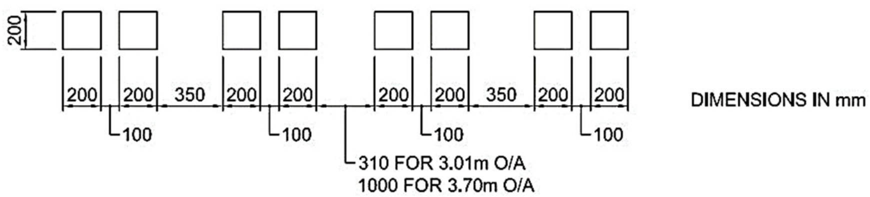



4. PRIME MOVER + 36 TONNE DOLLY + 144 TONNE 8-LINE PLATFORM
   (G.C.M. = 204 TONNES)

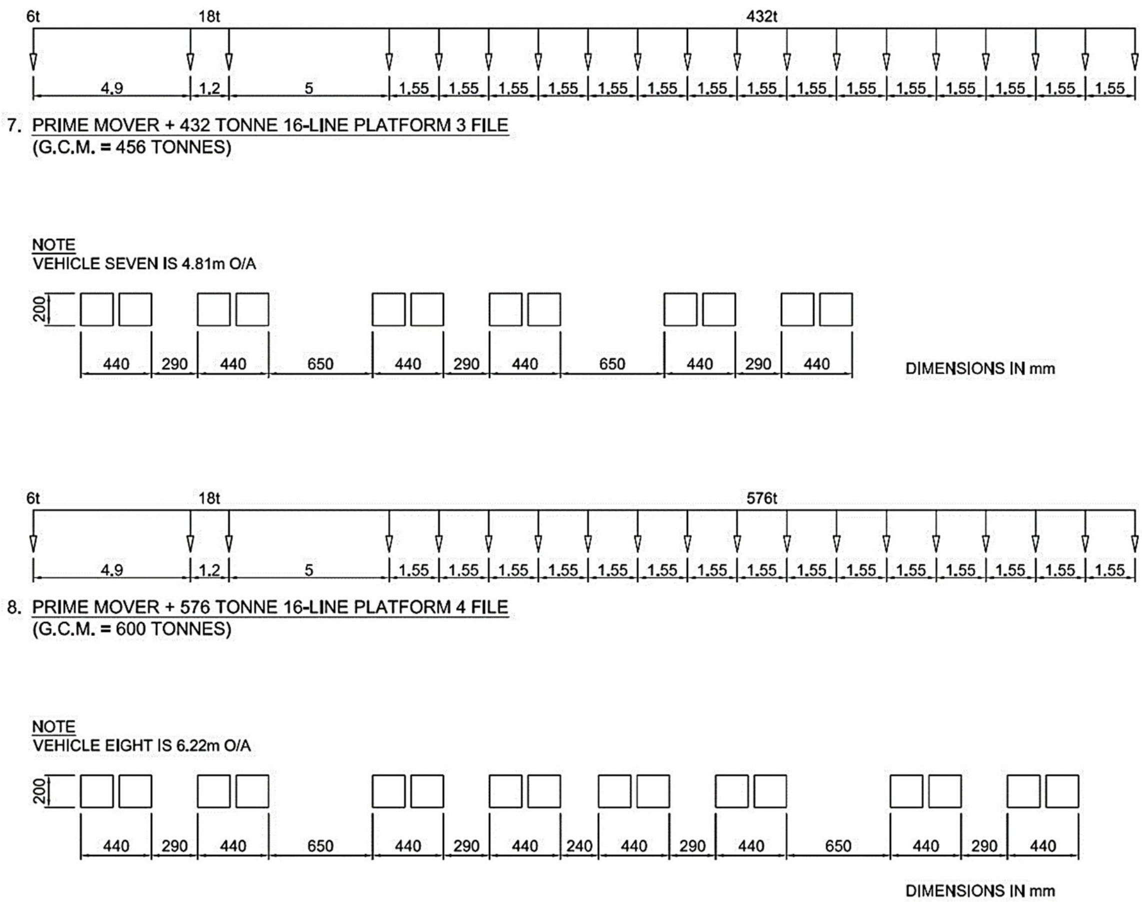



5. PRIME MOVER + 216 TONNE 12-LINE PLATFORM
   (G.C.M. = 240 TONNES)

NOTE
VEHICLES 1+2 CAN HAVE AN O/A WIDTH OF EITHER 3.01m OR 3.70m.
VEHICLE 5 IS 3.01m O/A
VEHICLE 4 IS 3.70m O/A




(a)




(b)

Figure 4.3 Magnitude, Spread and Axle Spacings of Loads for Group 2 Vehicles, (a) 8-Tyre Vehicles, (b) 12-Tyre & 16-Tyre Vehicles

For undivided bridges (and one carriageway bridges), the vehicle is to be placed up to ± 1.0 m from the bridge centreline and no other co-existing live loading is to be included on the bridge (see Figure 4.4(a)). There should be always a minimum clearance of 200 mm between the kerb and outside edge of the tyres.

For divided carriageway bridges, vehicles are to be placed up to ± 1.0 m from the carriageway centreline (in the direction of travel) with a minimum clearance of 200 mm maintained between the kerb and outside edge of the tyres (see Figure 4.4(b)). No other coincident live loading is to be included within the carriageway considered. 50% of the SM1600 loadings are to be applied in the other carriageway, positioned to give the worst load ratings. The accompanying lane factors shall be applied to these co-existing loadings in this carriageway in accordance with AS5100.2:2017, starting from 1.0 for first lane loaded with 50% of the SM1600. The vehicle positioning and dynamic load allowance of these 50% SM1600 loadings should be as per AS5100.7:2017 Cl. 11.3.

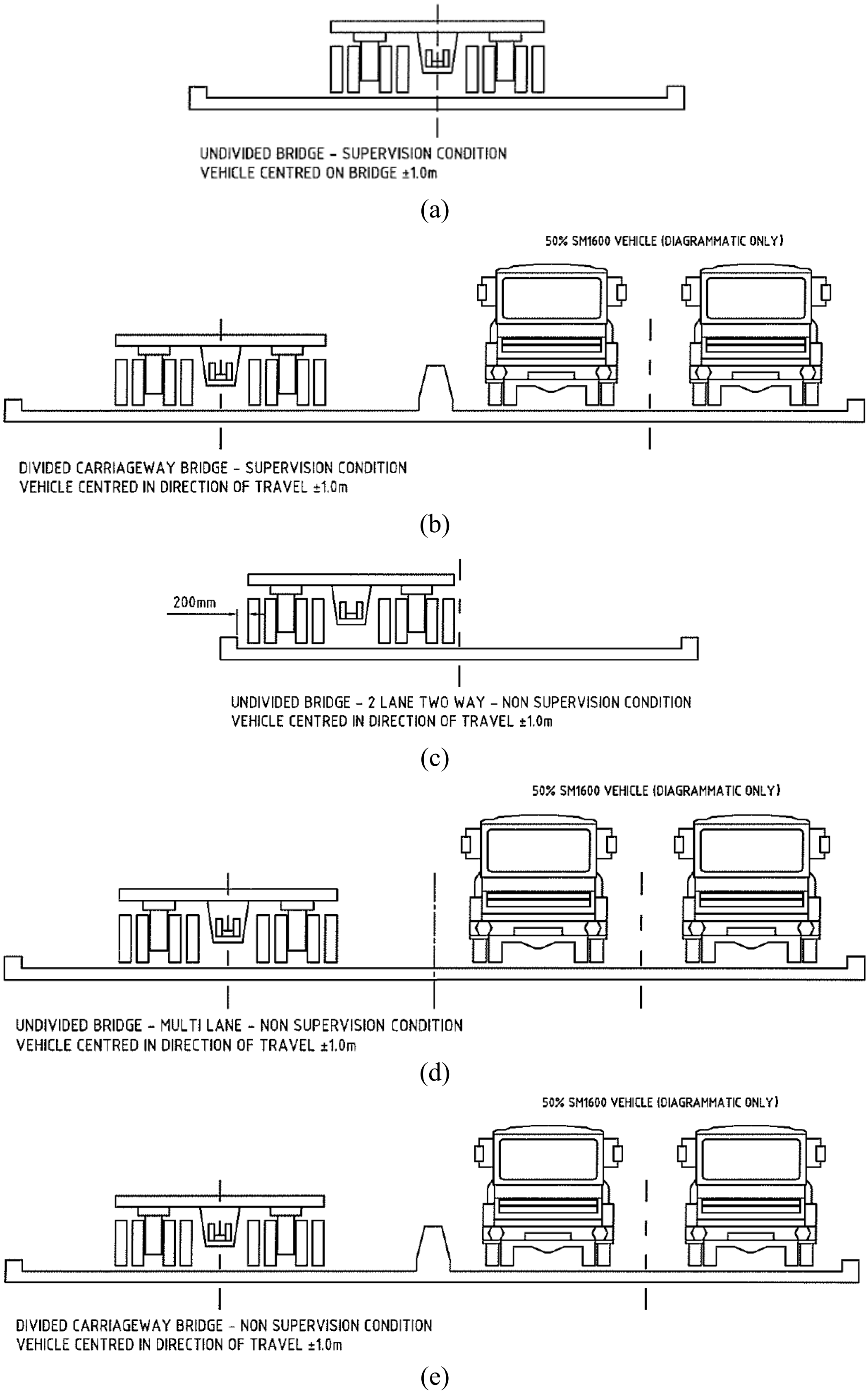



Figure 4.4 Example Vehicle Position for Load Rating of Group 2 Vehicles

### 4.4.6 Group 2 Vehicles (Non-Supervised)

Non-supervised load rating shall only be calculated for Group 2 Vehicle 4 and Group 2 Vehicle 5. The DLA of non-supervised Group 2 Vehicles shall be taken as 0.3. The positioning of Group 2 vehicles shall be the worst case of ignoring or considering temporary/ mountable median/verge/footpath/kerbs. Vehicles must be positioned within full width between external barriers on the bridge to ensure the bridge is rated for road traffic to allow for future changes to lane markings.

For undivided bridges, vehicles are to be placed up to ± 1.0 m from the carriageway centreline (in the direction of travel) with a minimum clearance of 200 mm between the kerb and outside edge of tyres (see Figure 4.4(c) and 4.4(d)). For bridges with a single lane in the direction of travel and kerb to kerb width less than or equal to 8.2m, no other coincident live loading is to be included. For multilane bridges or bridges with kerb-to-kerb width more than 8.2m, 50% of the SM1600 loadings are to be applied in the other carriageway, positioned to give the worst load ratings. The accompanying lane factors shall be applied to these co-existing loadings in this carriageway in accordance with AS5100.2:2017, starting from 1.0 for first lane loaded with 50% of the SM1600. The vehicle positioning and dynamic load allowance of these 50% SM1600 loadings should be as per AS5100.7:2017 Cl. 11.3.

For divided carriageway bridges, vehicles are to be placed up to ± 1.0 m from the carriageway centreline (in the direction of travel) with a minimum clearance of 200 mm maintained between the kerb and outside edge of the tyres (see Figure 4.4(e)). 50% of the SM1600 loadings are to be applied in the other carriageway, positioned to give the worst load ratings. The accompanying lane factors shall be applied to these co-existing loadings in this carriageway in accordance with AS5100.2:2017, starting from 1.0 for first lane loaded with 50% of the SM1600. The vehicle positioning and dynamic load allowance of these 50% SM1600 loadings should be as per AS5100.7:2017 Cl. 11.3.

Please note, no other co-existing Group 2/HLP vehicle should be considered in any adjacent lanes or in the same lane for Group 2 vehicles (both supervised and unsupervised) load rating.

## 4.5 CALCULATE PROJECTED FUTURE TRAFFIC LOAD DEMAND

Determining the nominal fatigue life of a bridge requires a fatigue assessment consisting of cumulative fatigue damage of the critical components of a bridge. For the purposes of this assessment, the cumulative fatigue damage shall be the sum of the damage due to historical loading as well as projected future traffic and the nominal fatigue life shall be considered to have been reached when the cumulative damage sums to unity. Thus, projected future traffic demand needs to be calculated for determining the remaining fatigue life of a bridge. As the projected demand will be used for calculating the number of stress cycles, this projection should be in terms of required standard vehicles as per Austroads Vehicle Classification or percentage of design vehicles specified in this section, unless otherwise specified by EBL. Please note, MRWA will provide future traffic demand data when it is not impossible to be calculated due to data access privilege.

## 4.6 PREPARE BRIDGE MODEL AND CALCULATE ACTION EFFECTS

Structural analysis and action effect determination shall be carried out in accordance with AS 5100 (2017) Bridge Design code and Structures Engineering Design Manual Document 3912/03. The whole of the structure shall be analysed as part of the load rating assessment. The superstructure is to be checked for both longitudinal and transverse load effects.

Moment re-distribution can be included in the rating analysis and the allowable percentage of moment calculated in accordance with AS 5100 (2017). However, for rating of existing bridges with member section capacities calculated from actual or assumed material properties, this allowable percentage is further limited as follows. The hog bending moments may be reduced, but only to the point where the hog and sag ratings balance, as there is no point in re-distributing hog moment to the point where the sag region rating becomes controlling. This limit, based on balancing the hog and sag ratings, may be less than the allowable percentage as determined by AS 5100 (2017). Further, it is important when undertaking re-distribution of moment that the rotational capacity of the section is not exceeded at a plastic hinge (i.e. rupture failure occurs). Therefore, the maximum amount of moment available for re-distribution is directly related to the ultimate section capacity and is limited to the allowable percentage as determined by AS 5100 (2017) multiplied by the member section capacity (NOT the vehicle design moment) at the specific support being assessed. The lesser value of the above two conditions is to be used in the determination of the ratings.

Simply supported bridge decks with up to 20 degrees skew may be modelled and analysed as square bridges with sufficient accuracy and continuous bridge decks with up to 10 degrees skew may be modelled and analysed as square bridges with sufficient accuracy.

For most structures, the effect of the kerb, or edge beam, may be ignored. If it is included it usually attracts a high moment, much more than it can carry, so it will only crack and re-distribute anyway. If the edge beam is stiff and heavily reinforced it may need to be considered, but an iterative approach may be required to assess the amount of load it attracts and balance this to its capacity.

Analysis of piers, columns, capbeams and bearings shall be in accordance with AS 5100 (2017). It is important to check the “as-is” condition and situation, as age, any out of plumb of the columns, or misplacement of bearings can considerably increase forces. Also, any deterioration, e.g. chloride attack at the base of columns or corrosion of steel columns etc., may need to be allowed for. Check for loads calculated as above, although if critical, it may also be necessary to include stream forces for substructure checks as per the code. This will generally be specified when required.

Foundations are only usually checked if there is some doubt about their condition, e.g. following scour from flooding. Any analysis that is deemed required shall be undertaken in accordance with AS 5100 (2017).

The introduction of the MCFT shear design in AS 5100.5:2017 has made the capacity calculation for concrete bridge section more dependent on the combined action effects of bending, shear, torsion, and axial force, as applicable. This change enhances the requirement on finding action effects for vehicles placed in the most onerous position as per Section 4.4, as the combined action assessment makes judgement of critical positions and actions less reliable. Accordingly, calculation of ULS load ratings at multiple closely spaced locations along a concrete bridge is necessary to correctly identify the critical load rating location (for example, checking at each 100 mm increments within the critical region) prior to reporting the least controlling load rating in terms of bending and shear.

## 4.7 CAPACITY AND LOAD RATING

### 4.7.1 Loads and Load Factors

The loadings and load factors (ULS and SLS) shall be based on the AS 5100 (2017) bridge design code with several important modifications as detailed below.

**Dead Load and Superimposed Dead Load**

For existing structures, it is theoretically possible to measure bridge components and obtain an accurate measure of actual self-weight. However, this is rarely done, and dimensions on the drawings are commonly used. A more accurate assessment of superimposed dead loads can be made though, especially the thickness of any surfacing. This should be measured during the inspection, unless noted on design documentation, or a value (say 50mm) may be assumed based on the agreement with EBL. Unless otherwise specified, the Load Factors for Superimposed Dead Load should be as per code specified for 'all structures'.

**Load**

Live Traffic loads should be considered as per AS 5100 (2017) and Section 4.4. The live load factor for the T44 design vehicle shall be used for all Group 1 Vehicles and the live load factor for HLP design vehicles shall be used for all Group 2 Vehicles. For special cases, historical design vehicles or nominated permit vehicles may be added for load rating after consultation and agreement with EBL.

**Differential Temperature**

Differential temperature effects should be considered as per AS 5100 (2017) code. Please note, differential temperature effects should not control the strength (ULS) rating of the structure for the T44 vehicle. That is, if the ULS ratings for the T44 vehicle fall below 100% when the effects of differential temperature are included, and where the bridge condition is good with no notable evidence of structural distress, consideration shall be given to excluding the differential temperature effects.

**Differential Settlement**

Differential settlement effects should be considered as per AS 5100 (2017) code. As these effects are long-term effects and will be reduced through creep, the long-term Elastic Modulus is to be used for all differential settlement effects calculation. In absence of accurate calculations, following values shall be adopted for the analysis of differential settlement effects:

| Foundation Condition | Abutment Settlement | Pier Settlement | Comments |
|---|---|---|---|
| Spread Footings (non-cohesive soils) | 10 mm | 20 mm | Free draining granular sands |
| Spread Footings (cohesive soils) | Varies | Varies | Requires assessment based on geotechnical data |
| Spread Footings (on sound rock) | 5 mm | 5 mm | If borelogs indicate sound rock, or rock bit refused, reduce to 0 |
| Piled Foundations | 5 mm | 5 mm | If borelogs indicate very dense soil or rock with SPT values > 120 at pile toe, reduce to 0 |

Note: for new bridges the settlement shall be derived using geotechnical analysis based on geotechnical investigation results.

For existing structures, a load factor of 1.0 shall be used for differential settlement effects at both the serviceability and ultimate limit states, whilst load factors should be in accordance with the AS 5100 (2017) code for new structures, ratings for the cases of including and excluding differential settlement shall be carried out.

**Shrinkage and Creep**

Both long-term shrinkage and creep losses must be considered (for both ULS and SLS) as per AS 5100 (2017) unless otherwise specified by EBL.

**Prestress Parasitics**

As per AS 5100 (2017).

**Other Loads**

Several loads, which may be important and have been considered for design, e.g., wind, flood, earthquake etc., are not considered for load rating assessments, except in exceptional circumstances (after the consultation and agreement with EBL). An assessment of the longitudinal braking forces shall be considered and carried out.

**Fatigue**

Fatigue assessment, if required by EBL/Region, needs to be carry out as per AS 5100 (2017) after the consultation and agreement with EBL.

### 4.7.2 Capacity and Load Rating Factor Calculation

The capacity assessment and load rating factor calculations should be as per AS 5100 (2017) Bridge Design Code and Senior Engineer Structures (SES) Circulars. Please note, prior to assessing the capacity, all possible failure modes should be identified.

Unless otherwise agreed with EBL, the extension of longitudinal reinforcement and tendons should be as per the para. 1 cl. 8.2.9.1 of AS 5100.5:2017 (incorporating amendments) for calculating the load rating (i.e., at every section, the additional longitudinal forces to be considered as caused by shear and torsion as specified in Cl. 8.2.7, Cl 8.2.8 and Fig. 8.2.9.1).

Where the transverse shear reinforcement changes ($A_{sv}$ or s), a pragmatic approach is to ignore the additional capacity gained through general detailing as per cl. 8.3.2.3 AS 5100.5:2017 (i.e., to ignore additional shear reinforcement area that gained through extending shear reinforcement in the direction of decreasing shear). A note should be added in the load rating report and design summary sheet, clarifying whether this approach has been adopted or not for calculating the load rating.

A detailed example for capacity and load rating factor calculation of a prestressed concrete bridge is given in Appendix B. Please note, Appendix B is provided as an example and for reference only; for each bridge rating the rating engineer shall perform the checks according to the best engineering judgment/practice to assess the most critical structural conditions that determine the rating.

### 4.7.3 Load Rating Reporting

The load rating of an existing bridge is generally reported as a load rating memo and report. A typical load rating memo and report for a non-timber bridge are provided in Appendix C.

For new bridge design, the load rating results should be reported as Design Summary Sheets (DSS). Typical DSS examples for non-timber bridges are provided in Section 2 of BBDIM.

The DSS incorporating load ratings for all design vehicles must be submitted at the IFC (i.e., Issued for Construction) stage to MRWA for review and acceptance by MRWA.

# APPENDIX A

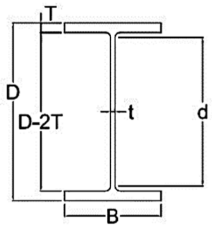
## Typical Steel Section Properties (Old Imperial Beams)

## APPENDIX A - TYPICAL STEEL SECTION PROPERTIES (OLD IMPERIAL BEAMS)

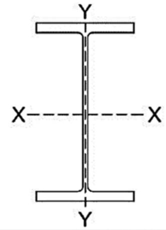
### Universal Beams

#### Properties for Designing

All universal beams produced in Australia have parallel flanges and the section properties listed are based on this geometry.




Moment of Inertia - I
Elastic Modulus - Z
Radius of Gyration - r

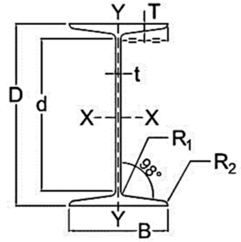



| Designation | | Depth of | Flange | | Web | Radius | Area of | Ratio | Depth |
|---|---|---|---|---|---|---|---|---|---|
| Nominal Size | Wt per Foot | Section D | Width B | Thickness T | Thickness t | Root r | Section | D/T | Between Fillets d |
| in | lb | in | in | in | in | in | $in^2$ | - | in |
| 30 x 10.5 | 196 | 31.22 | 10.836 | 1.461 | 0.900 | 0.65 | 57.49 | 21.4 | 27.00 |
| | 180 | 31.00 | 10.766 | 1.349 | 0.830 | 0.65 | 52.90 | 23.0 | 27.00 |
| | 164 | 30.76 | 10.694 | 1.232 | 0.768 | 0.65 | 48.16 | 25.0 | 27.00 |
| | 148 | 30.53 | 10.622 | 1.115 | 0.686 | 0.65 | 43.46 | 27.4 | 27.00 |
| | 132 | 30.30 | 10.551 | 1.000 | 0.615 | 0.65 | 38.87 | 30.3 | 27.00 |
| | 116 | 30.00 | 10.500 | 0.850 | 0.564 | 0.65 | 34.17 | 35.3 | 27.00 |
| | 99 | 29.68 | 10.444 | 0.690 | 0.508 | 0.65 | 29.15 | 43.0 | 27.00 |
| 27 x 10 | 114 | 27.28 | 10.070 | 0.932 | 0.570 | 0.60 | 33.57 | 29.3 | 24.22 |
| | 102 | 27.07 | 10.018 | 0.827 | 0.518 | 0.60 | 30.04 | 32.7 | 24.22 |
| | 94 | 26.91 | 9.990 | 0.747 | 0.490 | 0.60 | 27.69 | 36.0 | 24.22 |
| | 84 | 26.69 | 9.962 | 0.637 | 0.462 | 0.60 | 24.73 | 41.9 | 24.22 |
| 24 x 9 | 94 | 24.29 | 9.061 | 0.872 | 0.516 | 0.60 | 27.65 | 27.9 | 21.55 |
| | 84 | 24.09 | 9.015 | 0.772 | 0.470 | 0.50 | 24.74 | 31.2 | 21.55 |
| | 76 | 23.91 | 8.985 | 0.682 | 0.440 | 0.50 | 22.39 | 35.1 | 21.55 |
| | 68 | 23.71 | 8.961 | 0.582 | 0.416 | 0.50 | 20.02 | 40.7 | 21.55 |
| 21 x 8.25 | 82 | 21.44 | 8.342 | 0.840 | 0.502 | 0.50 | 24.16 | 25.5 | 18.76 |
| | 73 | 21.24 | 8.295 | 0.740 | 0.455 | 0.50 | 21.48 | 28.7 | 18.76 |
| | 68 | 21.13 | 8.270 | 0.685 | 0.430 | 0.50 | 20.04 | 30.8 | 18.76 |
| | 62 | 20.99 | 8.240 | 0.615 | 0.400 | 0.50 | 18.35 | 34.1 | 18.76 |
| | 55 | 20.80 | 8.216 | 0.520 | 0.376 | 0.50 | 16.19 | 40.0 | 18.76 |
| 18 x 7.5 | 66 | 18.40 | 7.592 | 0.770 | 0.450 | 0.40 | 19.42 | 23.9 | 16.06 |
| | 60 | 18.25 | 7.558 | 0.695 | 0.416 | 0.40 | 17.66 | 26.3 | 16.06 |
| | 55 | 18.12 | 7.532 | 0.630 | 0.390 | 0.40 | 16.20 | 28.8 | 16.06 |
| | 50 | 18.00 | 7.500 | 0.570 | 0.358 | 0.40 | 14.72 | 31.6 | 16.06 |
| | 45 | 17.86 | 7.476 | 0.500 | 0.334 | 0.40 | 13.24 | 35.7 | 16.06 |
| 16 x 7 | 50 | 16.25 | 7.073 | 0.628 | 0.380 | 0.40 | 14.72 | 25.9 | 14.19 |
| | 45 | 16.12 | 7.039 | 0.563 | 0.346 | 0.40 | 13.24 | 28.6 | 14.19 |
| | 40 | 16.00 | 7.000 | 0.503 | 0.307 | 0.40 | 11.19 | 31.8 | 14.19 |
| | 36 | 15.85 | 6.992 | 0.428 | 0.299 | 0.40 | 10.01 | 37.0 | 14.19 |

| About X-X | | | About Y-Y | | | Torsion | Ratio | | Designation | |
|---|---|---|---|---|---|---|---|---|---|---|
| I | Z | r | I | Z | r | Constant J | (D-2T)/t | (B-t)/2T | Wt per Foot | Nominal Size |
| $in^4$ | $in^3$ | in | $in^4$ | $in^3$ | in | $in^4$ | - | - | lb | in |
| 8790 | 563 | 12.36 | 312 | 57.5 | 2.33 | 31.6 | 31.44 | 3.40 | 196 | 30 x 10.5 |
| 8030 | 518 | 12.32 | 282 | 52.4 | 2.31 | 24.9 | 34.10 | 3.68 | 180 | |
| 7250 | 471 | 12.27 | 252 | 47.2 | 2.29 | 19.0 | 37.33 | 4.03 | 164 | |
| 6490 | 425 | 12.22 | 224 | 42.1 | 2.27 | 14.2 | 41.26 | 4.46 | 148 | |
| 5750 | 380 | 12.18 | 196 | 37.2 | 2.25 | 10.3 | 46.02 | 4.98 | 132 | |
| 4930 | 329 | 12.01 | 165 | 31.3 | 2.19 | 6.86 | 50.18 | 5.85 | 116 | |
| 4060 | 274 | 11.80 | 131 | 25.2 | 2.12 | 4.17 | 55.71 | 7.20 | 99 | |
| 4090 | 300 | 11.04 | 159 | 31.6 | 2.18 | 7.82 | 44.59 | 5.10 | 114 | 27 x 10 |
| 3610 | 267 | 10.96 | 139 | 27.7 | 2.15 | 5.62 | 49.07 | 5.74 | 102 | |
| 3270 | 243 | 10.87 | 124 | 24.9 | 2.12 | 4.35 | 51.87 | 6.36 | 94 | |
| 2840 | 212 | 10.70 | 105 | 21.1 | 2.06 | 3.03 | 55.01 | 7.46 | 84 | |
| 2690 | 221 | 9.86 | 108 | 23.9 | 1.98 | 5.53 | 43.69 | 4.90 | 94 | 24 x 9 |
| 2370 | 197 | 9.79 | 94.5 | 21.0 | 1.95 | 3.95 | 47.97 | 5.53 | 84 | |
| 2100 | 176 | 9.69 | 82.6 | 18.4 | 1.92 | 2.89 | 51.24 | 6.27 | 76 | |
| 1820 | 154 | 9.53 | 70.0 | 15.6 | 1.87 | 2.00 | 54.20 | 7.34 | 68 | |
| 1830 | 171 | 8.71 | 81.5 | 19.5 | 1.84 | 4.57 | 39.36 | 4.67 | 82 | 21 x 8.25 |
| 1600 | 151 | 8.64 | 70.6 | 17.0 | 1.81 | 3.22 | 43.43 | 5.30 | 73 | |
| 1480 | 140 | 8.60 | 64.7 | 15.7 | 1.80 | 2.61 | 45.95 | 5.72 | 68 | |
| 1330 | 127 | 8.54 | 57.5 | 13.9 | 1.77 | 1.97 | 49.40 | 6.37 | 62 | |
| 1140 | 110 | 8.40 | 48.2 | 11.7 | 1.72 | 1.34 | 52.55 | 7.54 | 55 | |
| 1100 | 119 | 7.52 | 56.3 | 14.8 | 1.70 | 3.07 | 37.47 | 4.64 | 66 | 18 x 7.5 |
| 986 | 108 | 7.47 | 50.1 | 13.3 | 1.68 | 2.31 | 40.43 | 5.14 | 60 | |
| 891 | 98.4 | 7.42 | 45.0 | 11.9 | 1.67 | 1.77 | 43.23 | 5.67 | 55 | |
| 802 | 89.1 | 7.38 | 40.2 | 10.7 | 1.65 | 1.34 | 47.10 | 6.27 | 50 | |
| 706 | 79.1 | 7.30 | 34.9 | 9.33 | 1.62 | 0.962 | 50.48 | 7.14 | 45 | |
| 657 | 80.8 | 6.68 | 37.1 | 10.5 | 1.59 | 1.61 | 39.46 | 5.33 | 50 | 16 x 7 |
| 585 | 72.5 | 6.64 | 32.8 | 9.32 | 1.57 | 1.18 | 43.34 | 5.94 | 45 | |
| 517 | 64.6 | 6.62 | 28.8 | 8.23 | 1.56 | 0.851 | 48.84 | 6.65 | 40 | |
| 448 | 56.5 | 6.50 | 24.4 | 6.99 | 1.52 | 0.594 | 50.15 | 7.82 | 36 | |

# ROLLED STEEL JOISTS

## DIMENSIONS AND PROPERTIES




| Reference Number | Size D X B Inches | Weight per foot lbs | PLASTIC MODULI OF SECTION | | STANDARD THICKNESS INCHES | | RADII INCHES | | Straight Portion of Web d Inches | Sectional Area Inches$^2$ | MOMENT OF INERTIA INCHES$^4$ | | RADII OF GYRATION INCHES | | ELASTIC MODULI OF SECTION | | D/T | Size D x B x Wt. per Foot |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis X-X | Axis Y-Y | Web t | Flange T | Root R1 | Toe R2 | | | Axis X-X | Axis Y-Y | Axis X-X | Axis Y-Y | Axis X-X | Axis Y-Y | | |
| | 24 x 7.5* | 100 | 254.6 | 29.01 | 0.64† | 1.011 | 0.73 | 0.36 | 20.25 | 29.44 | 2605.04 | 64.30 | 9.41 | 1.48 | 217.09 | 17.00 | 23.7 | 24 x 7.5 x 100 |
| ASB 126 | 24 x 7.5 | 95 | 245.6 | 28.11 | 0.57 | 1.011 | 0.73 | 0.36 | 20.25 | 27.94 | 2533.04 | 62.54 | 9.52 | 1.50 | 211.09 | 16.68 | 23.7 | 24 x 7.5 x 95 |
| ASB 125 | 22 x 7 | 75 | 177.2 | 19.96 | 0.50 | 0.834 | 0.69 | 0.34 | 18.68 | 22.06 | 1676.80 | 41.07 | 8.72 | 1.36 | 152.44 | 11.73 | 26.4 | 22 x 7 x 75 |
| ASB 124 | 20 x 6.5 | 65 | 141.8 | 16.84 | 0.45 | 0.820 | 0.65 | 0.32 | 16.80 | 19.12 | 1226.17 | 32.56 | 8.01 | 1.31 | 122.62 | 10.02 | 24.4 | 20 x 6.5 x 65 |
| ASB 123 | 18 x 6 | 55 | 108.1 | 13.24 | 0.42 | 0.757 | 0.61 | 0.30 | 15.03 | 16.18 | 841.76 | 23.63 | 7.21 | 1.21 | 93.53 | 7.88 | 23.8 | 18 x 6 x 55 |
| ASB 121 | 16 x 6 | 50 | 88.79 | 12.44 | 0.40 | 0.726 | 0.61 | 0.30 | 13.09 | 14.71 | 618.09 | 22.47 | 6.48 | 1.24 | 77.26 | 7.49 | 22.0 | 16 x 6 x 50 |
| ASB 120 | 15 x 6 | 45 | 75.20 | 11.15 | 0.38 | 0.655 | 0.61 | 0.30 | 12.25 | 13.24 | 491.91 | 19.87 | 6.10 | 1.23 | 65.59 | 6.62 | 22.9 | 15 x 6 x 45 |
| ASB 119 | 14 x 5.5 | 40 | 61.95 | 9.02 | 0.37 | 0.627 | 0.57 | 0.28 | 11.39 | 11.77 | 377.06 | 14.79 | 5.66 | 1.12 | 53.87 | 5.38 | 22.3 | 14 x 5.5 x 40 |
| ASB 118 | 13 x 5 | 35 | 50.28 | 7.26 | 0.35 | 0.604 | 0.53 | 0.26 | 10.53 | 10.30 | 283.51 | 10.81 | 5.25 | 1.03 | 43.52 | 4.33 | 21.5 | 13 x 5 x 35 |
| ASB 117 | 12 x 8 | 65 | 92.46 | 26.63 | 0.43 | 0.904 | 0.77 | 0.38 | 8.30 | 19.12 | 487.77 | 65.18 | 5.05 | 1.85 | 81.29 | 16.30 | 13.3 | 12 x 8 x 65 |
| ASB 116 | 12 x 5 | 30 | 39.68 | 6.01 | 0.33 | 0.507 | 0.53 | 0.26 | 9.74 | 8.83 | 206.93 | 8.77 | 4.84 | 1.00 | 34.49 | 3.51 | 23.7 | 12 x 5 x 30 |
| ASB 115 | 10 x 8 | 70 | 80.46 | 28.97 | 0.60 | 0.970 | 0.70 | 0.35 | 6.33 | 20.58 | 345.04 | 71.61 | 4.09 | 1.87 | 69.01 | 17.90 | 10.3 | 10 x 8 x 70 |
| ASB 114 | 10 x 8 | 55 | 65.65 | 22.59 | 0.40 | 0.783 | 0.77 | 0.38 | 6.60 | 16.18 | 288.69 | 54.74 | 4.22 | 1.84 | 57.74 | 13.69 | 12.8 | 10 x 8 x 55 |
| ASB 113 | 10 x 6 | 40 | 46.74 | 11.89 | 0.36 | 0.709 | 0.61 | 0.30 | 7.09 | 11.77 | 204.80 | 21.76 | 4.17 | 1.36 | 40.94 | 7.39 | 14.1 | 10 x 6 x 40 |
| ASB 112 | 10 x 4.5 | 25 | 28.07 | 4.84 | 0.30 | 0.505 | 0.49 | 0.24 | 7.84 | 7.35 | 122.34 | 6.49 | 4.08 | 0.94 | 24.47 | 2.88 | 19.8 | 10 x 4.5 x 25 |
| ASB 111 | 9 x 7 | 50 | 53.18 | 18.67 | 0.40 | 0.825 | 0.69 | 0.34 | 5.61 | 14.71 | 208.13 | 40.17 | 3.76 | 1.65 | 46.25 | 11.48 | 10.9 | 9 x 7 x 50 |
| ASB 110 | 9 x 4 | 21 | 20.85 | 3.51 | 0.30 | 0.457 | 0.45 | 0.22 | 7.03 | 6.18 | 81.13 | 4.15 | 3.62 | 0.82 | 18.03 | 2.07 | 19.7 | 9 x 4 x 21 |
| ASB 109 | 8 x 6 | 35 | 32.94 | 10.71 | 0.35 | 0.648 | 0.61 | 0.30 | 5.36 | 10.30 | 115.06 | 19.54 | 3.34 | 1.38 | 28.76 | 6.51 | 12.3 | 8 x 6 x 35 |
| ASB 108 | 8 x 4 | 18 | 16.02 | 2.99 | 0.28 | 0.398 | 0.45 | 0.22 | 6.15 | 5.30 | 55.63 | 3.51 | 3.24 | 0.81 | 13.91 | 1.75 | 20.1 | 8 x 4 x 18 |
| ASB 107 | 7 x 3.5 | 15 | 11.79 | 2.30 | 0.25 | 0.398 | 0.41 | 0.20 | 5.25 | 4.42 | 35.90 | 2.41 | 2.85 | 0.74 | 10.26 | 1.38 | 17.6 | 7 x 3.5 x 15 |
| ASB 106 | 6 x 5 | 25 | 17.50 | 6.48 | 0.33 | 0.561 | 0.53 | 0.26 | 3.71 | 7.35 | 45.16 | 9.88 | 2.48 | 1.16 | 15.05 | 3.95 | 10.7 | 6 x 5 x 25 |
| ASB 105 | 6 x 3 | 12 | 8.08 | 1.63 | 0.23 | 0.377 | 0.37 | 0.18 | 4.40 | 3.53 | 20.99 | 1.46 | 2.44 | 0.64 | 7.00 | 0.97 | 15.9 | 6 x 3 x 12 |
| ASB 104 | 5 x 2.5 | 9 | 5.06 | 1.03 | 0.20 | 0.347 | 0.33 | 0.16 | 3.60 | 2.65 | 10.91 | 0.79 | 2.03 | 0.55 | 4.36 | 0.63 | 14.4 | 5 x 2.5 x 9 |
| ASB 103 | 4 x 3 | 10 | 4.54 | 1.47 | 0.24 | 0.347 | 0.37 | 0.18 | 2.50 | 2.94 | 7.79 | 1.33 | 1.63 | 0.67 | 3.89 | 0.88 | 11.5 | 4 x 3 x 10 |
| ASB 102 | 4 x 1.75 | 5 | 2.15 | 0.37 | 0.17 | 0.239 | 0.27 | 0.13 | 2.86 | 1.47 | 3.66 | 0.19 | 1.58 | 0.36 | 1.83 | 0.21 | 16.7 | 4 x 1.75 x 5 |

* Nominal size – Actual width is 7 5/16 inches.
† Nominal sizse – Actual thickness is 0.6325 inches.

# APPENDIX B

## Example Load Rating of a Prestressed Concrete Tee-Roff Beam

## APPENDIX B - EXAMPLE LOAD RATING CALCULATION OF A PRESTRESSED CONCRETE TEE-ROFF BEAM

The following example illustrates the load rating of a prestressed concrete Tee-Roff beam for SM1600 design vehicle based on the AS 5100 (2017) code. This step by step process will help to understand the load rating process with known design vehicle action effects. The introduction of the modified compression field theory (MCFT) has made the bending and shear capacity as well as load rating calculation of a beam dependent on the applied loading action effects, i.e., dependent on bending, shear and torsional action effects. The load rating factor $k$, therefore, can be determined when scaling the live load action effects with $k$ results in the utilisation of the full capacity for any particular failure mechanism. The bending, shear and torsional action effects at full capacity utilisation will be,

$$M^* = M_{PE} + M_{CE} + k \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k \times T_{LL}^*$$

where,

$M_{PE}$ = Factored ultimate moment due to all permanent effects (PE)
$M_{CE}$ = Factored ultimate moment due to co-existing live load
$M_{LL}^*$ = Factored ultimate moment due to vehicle live load effects (including γ and DLA)
$V_{PE}$ = Factored ultimate shear due to all permanent effects (PE)
$V_{CE}$ = Factored ultimate shear due to co-existing live load
$V_{LL}^*$ = Factored ultimate shear due to vehicle live load effects (including γ and DLA)
$T_{PE}$ = Factored ultimate torsion due to all permanent effects (PE)
$T_{CE}$ = Factored ultimate torsion due to co-existing live load
$T_{LL}^*$ = Factored ultimate torsion due to vehicle live load effects (including γ and DLA)

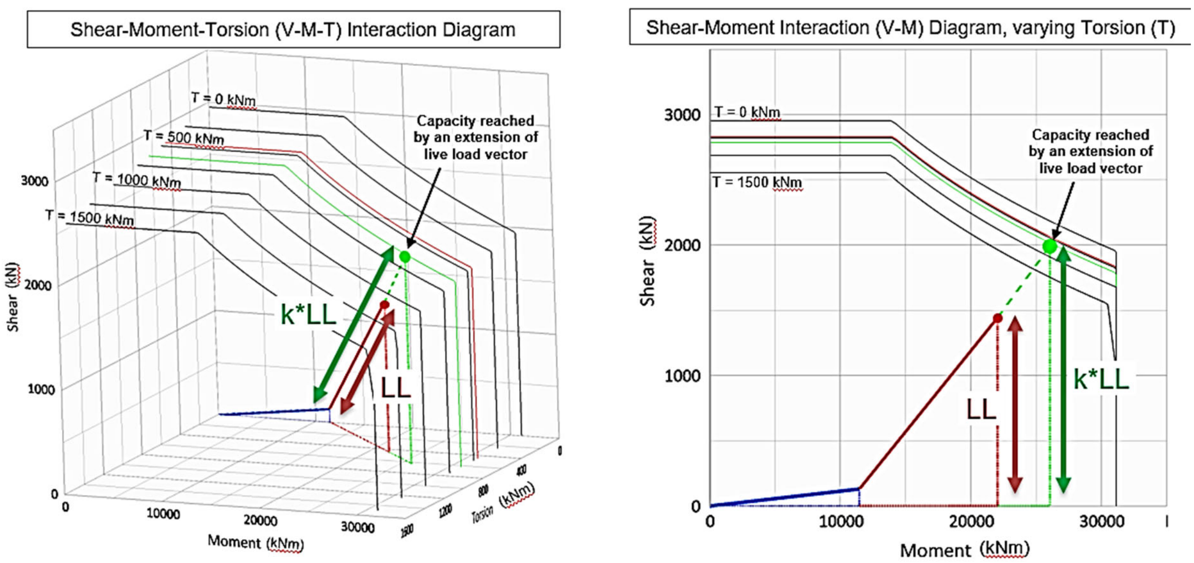



Figure B.1 Three-Dimensional Euclidian Capacity Surface Demonstrating Shear-Moment-Torsion (V-M-T) Interaction

Figure B.1 illustrates this interaction of bending, shear and torsion and the load rating factor $k$ can be interpreted as the scaling factor by which critical live load actions can be increased (or decreased), before the cross sectional capacity is reached.

Performing a beam/grillage analysis of a bridge can generate a huge number of combinations of $M_{LL}^*, V_{LL}^*$ $and$ $T_{LL}^*$ at a critical section due to moving vehicles position. Out of all these combinations, only three combination needs to be considered. They are,

(i) $M_{LL}^*$ max with corresponding $V_{LL}^*$ $and$ $T_{LL}^*$
(ii) $V_{LL}^*$ max with corresponding $M_{LL}^*$ $and$ $T_{LL}^*$
(iii) $T_{LL}^*$ max with corresponding $M_{LL}^*$ $and$ $V_{LL}^*$

The matrix below summarises the Ultimate Limit Sate (ULS) bending, shear and torsion cases at each cross-section that are required to be load rated as well as capacity calculated in the ultimate limit state load rating.

| Longitudinal Bending Scenarios | $M_{LL}^*$ Max (Corr. $V_{LL}^*$ & $T_{LL}^*$) | $V_{LL}^*$ Max (Corr. $M_{LL}^*$ & $T_{LL}^*$) | $T_{LL}^*$ Max (Corr. $M_{LL}^*$ & $V_{LL}^*$) |
|---|---|---|---|
| M1 | Y | N/A | N/A |
| M2 | Y | N/A | N/A |
| M3 | Y | N/A | N/A |
| Combined Shear and Torsion Scenarios | $M_{LL}^*$ Max (Corr. $V_{LL}^*$ & $T_{LL}^*$) | $V_{LL}^*$ Max (Corr. $M_{LL}^*$ & $T_{LL}^*$) | $T_{LL}^*$ Max (Corr. $M_{LL}^*$ & $V_{LL}^*$) |
| V1 | Y | Y | Y |
| V2 | Y | Y | Y |
| V3 | N/A | Y | Y |
| V4 | N/A | Y | Y |
| V5 | Y | Y | Y |

The load rating and capacity calculations should be performed at longitudinal cross-sections of the beam spaced at reasonably small increments (unless not required at compression fan regions) including at salient points such as changes of cross section or reinforcement. The calculations shown in this example have been performed for only one cross section for each scenario for demonstration purposes. Refer Figure B.2 for the cross section at mid-span of Span 1.

Also, no temperature load or support settlement is considered in this example. Furthermore, it is assumed for this example that all other failure modes such as transverse failure, interface shear, anchorage zone etc. are checked and load ratings are adequately above the longitudinal bending and shear rating factors.

## B.1 DETERMINE SECTION AND MATERIAL PROPERTIES

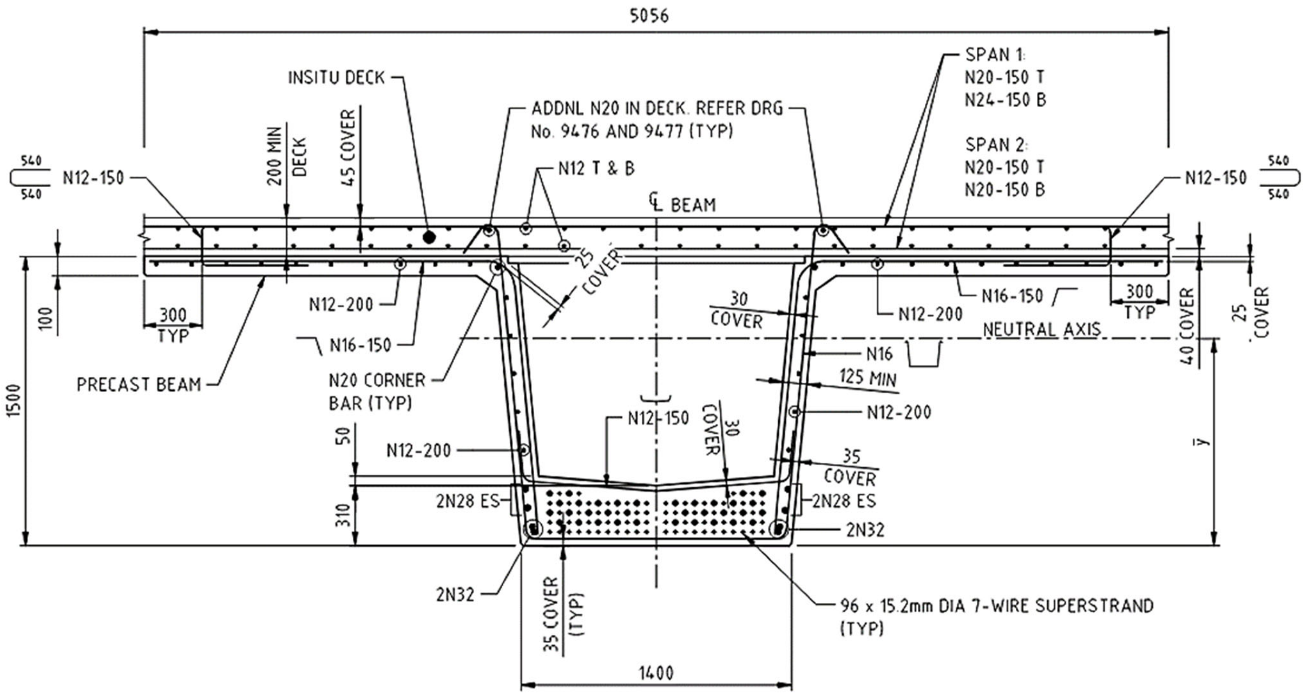



Figure B.2 Beam Cross Section at Mid-Span of Span 1

### Material Properties

| | |
|---|---|
| Concrete strength (Deck), $f'_{c_{deck}}$ | 40 MPa |
| Concrete strength (Beam), $f'_{c_{beam}}$ | 50 MPa |
| Modulus of elasticity (Deck), $E_{c\_Deck}$ | 32,800 MPa (AS 5100.5:2017 Table 3.1.2) |
| Modulus of elasticity (Beam), $E_{c\_Beam}$ | 34,800 MPa (AS 5100.5:2017 Table 3.1.2) |
| Yield strength (D500N), $f_{sy}$ | 500 MPa (AS 5100.5:2017 Table 3.2.1) |
| Modulus of elasticity (D500N), $E_s$ | 200x$10^3$ MPa (AS 5100.5: 2017 Clause 3.2.2) |
| Breaking strength (Strand), $f_{pb}$ | 1830 MPa (AS 5100.5:2017 Table 3.2.1) |

### Reinforcement Details

The number of layers and area of the D500N reinforcement within the composite cross section are presented below in Table B.1.

Table B.1 D500N Reinforcement within the Composite Cross Section

| Layer | $A_{st}$ (mm$^2$) | Depth from top of section (mm) |
|---|---|---|
| 1 | 2857 | 71 |
| 2 | 628 | 75 |
| 3 | 2857 | 130 |
| 4 | 2260 | 247 |
| 5 | 628 | 260 |
| 6 | 226 | 445 |
| 7 | 226 | 646 |
| 8 | 226 | 847 |
| 9 | 226 | 1038 |
| 10 | 226 | 1242 |
| 11 | 1232 | 1400 |
| 12 | 1232 | 1495 |
| 13 | 1608 | 1593 |
| 14 | 1608 | 1625 |

The number of layers and area of the prestressing strand within the typical composite cross section are presented below in Table B.2.

Table B.2 Prestressing Strand within the Typical Composite Cross Section

| Layer | $A_{st}$ (mm$^2$) | Depth from top of section (mm) |
|---|---|---|
| 1 | 1144 | 1430 |
| 2 | 3146 | 1480 |
| 3 | 3146 | 1530 |
| 4 | 3146 | 1580 |
| 5 | 3146 | 1630 |

## B.2 $\Phi M_u$ CALCULATION

**Calculate Transformed Flange Width**

The depth to the neutral axis is unknown however, it is assumed to be located in the top flange of the beam i.e. below the deck level. Following transformed flange widths were used to calculate the position of the neutral axis at ULS and SLS loading (note: this approximate transformation is only for identifying the neutral axis when it is below deck level and not for any other calculations).

As the deck of the composite section has a different 28-day compressive strength to that of the precast beam, in the ULS analysis, the width of the top flange is transformed as follows,

$$Transformed\ Width_{ULS} = \frac{f'_{c_{deck}}}{f'_{c_{beam}}} \times Top\ flange\ width = \frac{40}{50} \times 5056 = 4045\ mm$$

This is different to the SLS analysis where the width of the top flange is transformed based on the modular ratio of the elastic modulii of the deck concrete 28-day strength against that of the precast beam,

$$Transformed\ Width_{SLS} = \frac{E_{c\_Deck}}{E_{c\_Beam}} \times Top\ flange\ width = \frac{32{,}800}{34{,}800} \times 5056 = 4765\ mm$$

**Calculate $\boldsymbol{\Phi_b M_u}$**

The unfactored moment capacity $M_u$ (38115 kNm) was calculated based on the maximum stress reached in tendons at ultimate strength using Eq. 8.1.7(1) of AS 5100.5:2017 (alternatively stress-strain curve can be used in accordance with BBDIM Section 2). Solving for the neutral axis depth, it was determined that the neutral axis is located 203.6 mm from the top of the composite section which places it in the beam top flange as assumed. From this we determine that $\gamma d_n$ = 142.5 mm from the top of the composite section ($\gamma = 0.7$ and $\alpha_2 = 0.85$ are assumed conservatively considering neutral axis is below the deck level) and,

$$k_{uo} = \frac{d_n}{d_o} = \frac{203.6}{1630} = 0.125$$

Which gives a $\Phi_b$ for bending of 0.8 calculated in accordance with AS 5100.5:2017 Table 2.3.2 for item (b) Bending without axial tension or compression. This gives a factored ultimate beam bending capacity at mid-span of span 2,

$$\Phi_b M_u = 30492\ kNm$$

Please note, the $\Phi_b M_u$ for the example beam varies along the length due to the varied flange width. The above calculation is an example for determining $\Phi_b M_u$ at 16.96 m from the centreline of the Abutment 1 Bearing (approximately at the mid-span of span 1).

## B.3 CALCULATE EFFECTIVE SHEAR DEPTH $\mathbf{d_v}$

AS 5100.5:2017 Cl. 8.2.1.9 defines the effective shear depth ($d_v$) as "the distance between the resultants of the tensile and compressive forces due to flexure in Clause 8.1.2 but not less than the greater of $0.72D$ or $0.9d$, where $d$ is taken as the distance from the extreme compression fibre to the centroid of the longitudinal tension reinforcement in the half-depth of the section containing the flexural tension zone".

From the calculations above the distance between the resultants of the tensile and compressive forces due to flexure in Clause 8.1.2 is determined to be $1439\ mm$.

Similarly, the distance $d$ from the extreme compression fibre to the centroid of the longitudinal tension reinforcement in the half-depth of the section containing the flexural tension zone is determined to be 1543 $mm$ which gives,

$0.9d = 0.9 \times 1543 = 1387\ mm$
$0.72D = 0.72 \times 1700 = 1224\ mm$

The lower limit on $d_v$ is then $Maximum(1387, 1224) = 1387\ mm$
Thus, $d_v$ is calculated at 16.96 m from the centreline of the Abutment 1 Bearing (approximately at the mid-span of span 1) to be $Maximum(1387, 1439) = 1439\ mm$

## B.4 0.25 $\Phi T_{cr}$ CALCULATION

Refer AS 5100.5:2017 equation 8.2.1.2(2) reproduced below:

$$T_{cr} = 0.33\sqrt{f_c'}\frac{A_{cp}^2}{u_c}\sqrt{\left(1 + \frac{\sigma_{cp}}{0.33\sqrt{f_c'}}\right)}$$

$A_{cp}$ is diagrammatically (ignoring overhanging flanges) shown in Figure B.3 below:

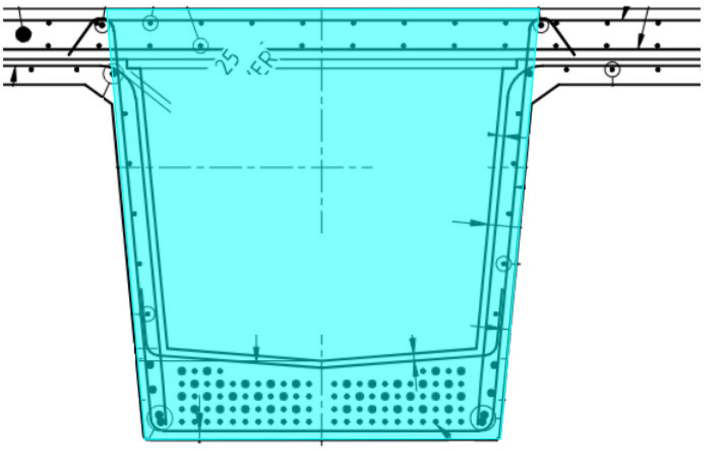



Figure B.3 Diagrammatic Representation of Torsional Parameter $A_{cp}$

And is determined from the following calculation:
$A_{cp} = b_f D + 2 \times 0.5 \times D \times D \times \tan(\theta_w)$
$= 1.4 \times 1.7 + 2 \times 0.5 \times 1.7 \times 1.7 \times \tan(5.45^\circ)$
$= 2.38 + 0.276 = 2.656\ m^2$
Where:
$b_f$ = Bottom flange width
$D$ = Overall depth of the composite section
$\theta_w$ = web angle to the vertical

$u_c$ is simply the perimeter of this shape and is calculated by the following equation:

$$u_c = 2b_f + 2 \times D \times \tan(\theta_w) + 2 \times \sqrt{D^2 + \left(D \times \tan(\theta_w)\right)^2}$$

$$= 2 \times 1.4 + 2 \times 1.7 \times \tan(5.45) + 2 \times \sqrt{1.7^2 + \left(1.7 \times \tan(5.45^\circ)\right)^2}$$

$$= 2.8 + 0.324 + 2 \times 1.708 = 6.54\ m$$

Note that AS 5100.5:2017 equation 8.2.1.2(3) places a limit on $\frac{A_{cp}^2}{u_c}$ for cellular structures such as this configuration of beam and slab such that

$$\frac{A_{cp}^2}{u_c} \leq 2A_o b_v$$

Where:
$A_o$ = area enclosed by shear flow path, including any areas of holes therein
$b_v$ = effective width of the critical web, 125 mm at this location.

$A_o$ is defined by the mid-depth of the bottom flange, webs and the deck and is shown diagrammatically in Figure B.4 below:

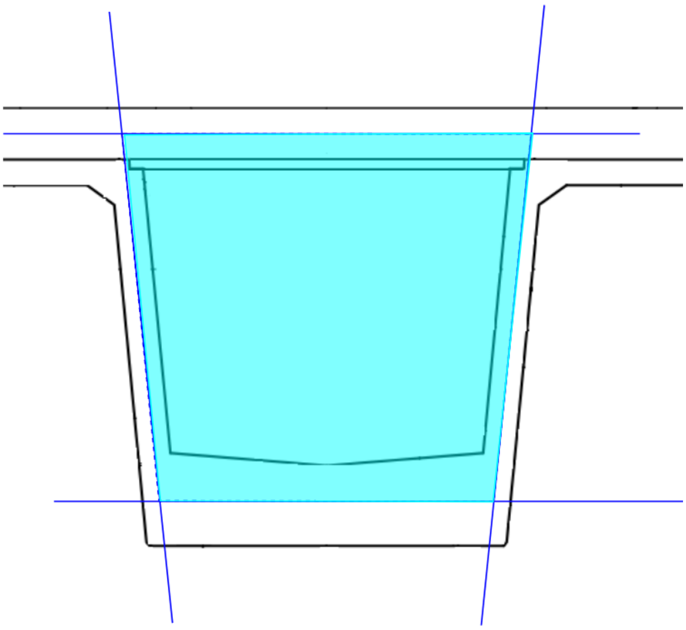


Figure B.4 Diagrammatic Representation of Torsional Parameter $A_o$

$A_o$ is determined to be 2.084 $m^2$ based on a bottom flange thickness of 310 mm, a nominal deck thickness of 200 mm at this location and web thicknesses of 125 mm respectively. Substituting these values into equation 8.2.1.2(3) we have,

$$\frac{A_{cp}^2}{u_c} \leq 2A_o b_v = \frac{2.656^2}{6.54} \leq 2 \times 2.084 \times 0.125 = 1.07 \leq 0.521 = 0.521$$

$\sigma_{cp}$ is defined as the average intensity of effective prestress in concrete at the centroid, or at the junction of the web and flange when the centroid lies inside the flange.

In this instance, the centroid of the concrete cross section is near the mid-depth of the composite section and as such $\sigma_{cp}$ is taken at this location.

$\sigma_{cp} = 6.1567\ MPa$

Substituting these values into the equation for $T_{cr}$ we have,

$$T_{cr} = 0.33\sqrt{50} \times 0.521 \times \sqrt{\left(1 + \frac{6.1567}{0.33\sqrt{50}}\right)} = 2319\ kNm$$

And $\Phi_t T_{cr} = 0.7 \times 2391 = 1624\ kNm$

And the limit in AS 5100.5:2017 equation 8.2.1.2(1) for when torsional effects are to be considered at 16.96 m from the centreline of the Abutment 1 Bearing (approximately at the mid-span of span 1),

$0.25\Phi_t T_{cr} = 0.25 \times 1674 = 405.9\ kN$

## B.5 CALCULATE LOAD RATING FACTOR $k_{M1}$ FOR SCENARIO M1

**Scenario M1**

The load rating factor $k_{M1}$ for scenario M1 is the factor of live loads that results in full utilisation of the bending capacity of the cross section under consideration. The objective function is,

$\Phi_b M_u = M_{PE} + M_{CE} + k_{M1} \times M_{LL}^*$

This equation needs to be solved for $k_{M1}$.

**Location**

16.96 m from the centreline of the abutment 1 bearing (approximately at the mid-span of span 1) with beam flange width 5056 mm.

**Section Parameters**

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1439\ mm$ | $A_{sv} = 402\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1439\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 541\ mm$ | $\alpha_v = 90°$ | $A_o = 2.084 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 250\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.363 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 125\ mm$ | $A_{st.D/2} = 6132\ mm^2$ | $u_h = 6169\ mm$ |
| $d = 1543\ mm$ | $\sigma_{cp} = 6.157\ MPa$ | $A_{pt.D/2} = 13728\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 203.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1974 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt}\sigma_{pt} = 26480 \times 10^3 N$ |
| $k_{uo} = 0.125$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7 = 1281\ MPa$ | $\Phi M_u = 30492 \times 10^6\ Nmm$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25\Phi T_{CR} = 405.9 \times 10^6\ Nmm$ |

**Action Effects**

| | | |
|---|---|---|
| $M_{PE} = 11659 \times 10^6 Nmm$ | $V_{PE} = 92 \times 10^3 N$ | $T_{PE} = 239 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M^*_{LL} = 18320 \times 10^6 Nmm$ | $V^*_{LL} = 9 \times 10^3 N$ | $T^*_{LL} = 319 \times 10^6 Nmm$ |

***Solve for $k_{M1}$***

Solving for $k_{M1}$ we determine its value to be 1.03 or 103%.

## B.6 CALCULATE LOAD RATING FACTOR $k_{M2}$ FOR SCENARIO M2

**Scenario M2**

The load rating factor $k_{M2}$ for scenario M2 is the factor of live loads that results in the utilisation of the total longitudinal reinforcement on the flexural tensile side. The objective function is,

for shear only,

$$\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{(M_{PE} + M_{CE} + k_{M2} \times M^*_{LL})}{\Phi_b z} + \frac{0.5 \times N^*}{\Phi_n} +$$

$$\left[\left(\frac{(V_{PE} + V_{CE} + k_{M2} \times V^*_{LL}) - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]\cot(\theta_v)$$

for combined shear and torsion,

$$\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{(M_{PE} + M_{CE} + k_{M2} \times M^*_{LL})}{\Phi_b z} + \frac{0.5 \times N^*}{\Phi_n} +$$

$$\sqrt{\left[\left(\frac{(V_{PE} + V_{CE} + k_{M2} \times V^*_{LL}) - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]^2 + \left(\frac{0.45 * (T_{PE} + T_{CE} + k_{M2} \times T^*_{LL}) \times u_h}{2A_o\Phi_t}\right)^2}\ \cot(\theta_v)$$

where,

$$V_{us} = \frac{A_{sv} f_{sy} d_v}{s}(\sin(\alpha_v)\cot(\theta_v) + \cos(\alpha_v))$$

$$\left[\left(\frac{(V_{PE} + V_{CE} + k_{M2} \times V^*_{LL}) - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right] \geq 0$$

and,

z is the internal moment lever arm between the centroids of the flexural compression force and the flexural tension force acting on the section, similar to $d_v$ but without the limits 0.72D or 0.9d. Please note, the approach to use of z as the internal moment lever arm is an approximate method. This approximation should NOT be used while designing and subsequently for the load rating of the Design Vehicles as the quantity of additional longitudinal reinforcements and tendons are known. An iterative method should be used for more accurate rating calculation.

In this equation, $f_s$ should not exceed $f_{sy}$ and $\sigma_{pt}$ needs to be determined from the tendon stress-stain relationship based on bending strain compatibility.

This equation needs to be solved for $k_{M2}$. As the value of $\theta_v$ in the objective function depends on the action effects, it is also dependent on the value $k_{M2}$. The value of $k_{M2}$, therefore, needs to be determined using appropriate iterative solvers (for example using *goalseek* function). Furthermore, according to AS 5100.5:2017 Cl. 8.2.4, longitudinal strain $\varepsilon_x$ needs to be calculated based on one of the four alternative equations determined by their sign and magnitude of the torsional action effect $T^*$. But, the value of $T^*$ is also dependent on $k_{M2}$. Thus, four separates $k_{M2}$ needs to be calculated through iterative processes for following four alternative cases. The final value of $k_{M2}$, then, needs to be selected based on the sign of $\varepsilon_x$ and magnitude of $T^*$ compared against $0.25\Phi T_{CR}$. Please note, as per AS 5100.5:2017 Cl. 8.2.4.3 and 8.2.4.4, for sections closer than $d_o$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_o$ from the face of the support may be used in evaluating $\varepsilon_x$ and $k_v$. Furthermore, as per AS 5100.5:2017 Cl. 8.2.9, the regions adjacent to maximum moment not subject to significant torsion where the support or load introduces direct compression and a fan-shaped pattern of compressive stresses radiating from the point load or the support, the M2 (and V3) rating may be omitted, provided that extension of longitudinal reinforcement and tendons are compliant with this clause.

***Case 1 – shear only and $\boldsymbol{\varepsilon_x} \leq \mathbf{3.0} \times \mathbf{10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M2} \times V_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_sA_{st} + E_pA_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{M^*}{\Phi_b z} + \frac{0.5 \times N^*}{\Phi_n} + \left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]\cot(\theta_v)$$

***Case 2 – shear only and $\boldsymbol{\varepsilon_x} \geq -\mathbf{0.2} \times \mathbf{10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{M^*}{\Phi_b z} + \frac{0.5 \times N^*}{\Phi_n} + \left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]\cot(\theta_v)$$

***Case 3 – combined shear & torsion and $\boldsymbol{\varepsilon_x \leq 3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{M^*}{\Phi_b z} + \frac{0.5N^*}{\Phi_n} + \sqrt{\left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]^2 + \left(\frac{0.45 \times T^* \times u_h}{2A_o\Phi_t}\right)^2}\ \cot(\theta_v)$$

***Case 4 – combined shear & torsion and $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M2} \times V_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) = \frac{M^*}{\Phi_b z} + \frac{0.5N^*}{\Phi_n} + \sqrt{\left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]^2 + \left(\frac{0.45 \times T^* \times u_h}{2A_o \Phi_t}\right)^2} \cot(\theta_v)$$

**Location**

16.96 m from the centreline of the abutment 1 bearing (approximately at the mid-span of span 1) with beam flange width 5056 mm.

**Section Parameters**

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1439\ mm$ | $A_{sv} = 402\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1439\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 541\ mm$ | $\alpha_v = 90°$ | $A_o = 2.084 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 250\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.363 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 125\ mm$ | $A_{st.D/2} = 6132\ mm^2$ | $u_h = 6169\ mm$ |
| $d = 1543\ mm$ | $\sigma_{cp} = 6.157\ MPa$ | $A_{pt.D/2} = 13728\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 203.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1974 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt}\sigma_{pt} = 26480 \times 10^3 N$ |
| $k_{uo} = 0.125$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7 = 1281\ MPa$ | $\Phi M_u = 30492 \times 10^6\ Nmm$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25\Phi T_{CR} = 405.9 \times 10^6\ Nmm$ |

**Action Effects**

| | | |
|---|---|---|
| $M_{PE} = 11659 \times 10^6 Nmm$ | $V_{PE} = 92 \times 10^3 N$ | $T_{PE} = 239 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M_{LL}^* = 18320 \times 10^6 Nmm$ | $V_{LL}^* = 9 \times 10^3 N$ | $T_{LL}^* = 319 \times 10^6 Nmm$ |

**Solve for $k_{M2}$**

| | Case 1 | Case 2 | Case 3 | Case 4 |
|---|---|---|---|---|
| $k_{M2}$ solved using iteration | 1.03 | 1.03 | 0.98 | 0.97 |
| $\varepsilon_x$ | $0.474 \times 10^{-3}$ | $0.074 \times 10^{-3}$ | $0.471 \times 10^{-3}$ | $0.072 \times 10^{-3}$ |
| $\varepsilon_x$ sign check | Selected | Ignored | Selected | Ignored |
| $k_{M2}$ selected | 1.03 | | 0.98 | |
| $T^*$ | $567 \times 10^6$ N-mm | | $550 \times 10^6$ N-mm | |
| Check $T^* > 0.25\Phi T_{CR}$ | Ignored* | | Selected | |
| $k_{M2}$ | 0.98 or 98% | | | |

* If selected, the T* (calculated value for rating vehicle factored with selected $k_{M2}$) should be re-calculated as an uncracked sectional analysis and the re-check as per Eq. 8.2.1.2(1) of AS5100.5:2017. If torsion needs to be considered, ignore Case 1/2 and select Case 3/4.

## B.7 CALCULATE LOAD RATING FACTOR $k_{M3}$ FOR SCENARIO M3

**Scenario M3**

The load rating factor $k_{M3}$ for scenario M3 is the factor of live loads that results in the utilisation of the total longitudinal reinforcement on the flexural compressive side. The objective function is,

$$\frac{M_u}{z} - \left| \frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc} f_s \right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z} + \frac{0.5 \times N^*}{\Phi_n}$$

Now, replacing $\frac{M_u}{z}$ with $\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt} \sigma_{pt}\right)$ considering tensile and compressive force are equal, we get updated objective function,

$$\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt} \sigma_{pt}\right) - \left| \frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc} f_s \right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b Z} + \frac{0.5 \times N^*}{\Phi_n}$$

where,

$$\Delta A_{sc} = \frac{\Delta F_{cd}}{\Phi f_{sy}}$$

for shear only,

$$\frac{\Delta F_{cd}}{\Phi} = \left[ \left( \frac{(V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*) - \gamma_p P_v}{\Phi_s} \right) - 0.5 V_{us} \right] \cot(\theta_v) + \frac{0.5 * N^*}{\Phi_n} - \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z} \geq 0$$

for combined shear and torsion,

$$\frac{\Delta F_{cd}}{\Phi} = \sqrt{\left[ \left( \frac{(V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*) - \gamma_p P_v}{\Phi_s} \right) - 0.5 V_{us} \right]^2 + \left( \frac{0.45 * (T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*) \times u_h}{2 A_o \Phi_t} \right)^2} \cot(\theta_v)$$

$$+ \frac{0.5 \times N^*}{\Phi_n} - \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z} \geq 0$$

where,

$$V_{us} = \frac{A_{sv} f_{sy} d_v}{s} (\sin(\alpha_v) \cot(\theta_v) + \cos(\alpha_v))$$

$$\left[ \left( \frac{(V_{PE} + V_{CE} + k_{M2} \times V_{LL}^*) - \gamma_p P_v}{\Phi_s} \right) - 0.5 V_{us} \right] \geq 0$$

and,

z is the internal moment lever arm between the centroids of the flexural compression force and the flexural tension force acting on the section, similar to $d_v$ but without the limits 0.72D or 0.9d. Please note, the approach to use of z as the internal moment lever arm is an approximate method. This approximation should NOT be used while designing and subsequently for the load rating of the Design Vehicles as the quantity of additional longitudinal reinforcements and tendons are known. An iterative method should be used for more accurate rating calculation.

This equation needs to be solved for $k_{M3}$. As the value of $\theta_v$ in the objective function depends on the action effects, it is also dependent on the value $k_{M3}$. The value of $k_{M3}$, therefore, needs to be determined through using appropriate iterative solvers (for example using *goalseek* function). Furthermore, according to AS 5100.5:2017 Cl. 8.2.4, longitudinal strain $\varepsilon_x$ needs to be calculated based on one of the four alternative equations determined by their sign and magnitude of the torsional action effect $T^*$. But, the value of $T^*$ is also dependent on $k_{M3}$. Thus, four separates $k_{M3}$ needs to be calculated through iterative processes for following four alternative cases. The final value of $k_{M3}$, then, needs to be selected based on the sign of $\varepsilon_x$ and magnitude of $T^*$ compared against $0.25\Phi T_{CR}$. Please note, as per AS 5100.5:2017 Cl. 8.2.4.3 and 8.2.4.4, for sections closer than $d_o$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_o$ from the face of the support may be used in evaluating $\varepsilon_x$ and $k_v$.

***Case 1 – shear only and $\boldsymbol{\varepsilon_x \leq 3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2(E_sA_{st} + E_pA_{pt})}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\frac{\Delta F_{cd}}{\Phi} = \left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]\cot(\theta_v) + \frac{0.5 \times N^*}{\Phi_n} - \frac{M^*}{\Phi_b z} \geq 0$$

$$\Delta A_{sc} = \frac{\Delta F_{cd}}{\Phi f_{sy}}$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) - \left|\frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc}f_s\right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z}$$

***Case 2 – shear only and $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} * T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\frac{\Delta F_{cd}}{\Phi} = \left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]\cot(\theta_v) + \frac{0.5 \times N^*}{\Phi_n} - \frac{M^*}{\Phi_b z} \geq 0$$

$$\Delta A_{sc} = \frac{\Delta F_{cd}}{\Phi f_{sy}}$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) - \left|\frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc}f_s\right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z}$$

***Case 3 – combined shear & torsion and $\boldsymbol{\varepsilon_x} \leq \mathbf{3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\frac{\Delta F_{cd}}{\Phi} = \sqrt{\left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]^2 + \left(\frac{0.45 \times T^* \times u_h}{2A_o \Phi_t}\right)^2}\cot(\theta_v) + \frac{0.5 \times N^*}{\Phi_n} - \frac{M^*}{\Phi_b z} \geq 0$$

$$\Delta A_{sc} = \frac{\Delta F_{cd}}{\Phi f_{sy}}$$

$$\left(\sum_{i=1}^{n} A_{st}f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) - \left|\frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc}f_s\right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z}$$

***Case 4 – combined shear & torsion and*** $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$

$$M^* = M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{M3} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{M3} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\frac{\Delta F_{cd}}{\Phi} = \sqrt{\left[\left(\frac{V^* - \gamma_p P_v}{\Phi_s}\right) - 0.5V_{us}\right]^2 + \left(\frac{0.45 \times T^* \times u_h}{2A_o \Phi_t}\right)^2} \cot(\theta_v) + \frac{0.5 \times N^*}{\Phi_n} - \frac{M^*}{\Phi_b z} \geq 0$$

$$\Delta A_{sc} = \frac{\Delta F_{cd}}{\Phi f_{sy}}$$

$$\left(\sum_{i=1}^{n} A_{st} f_s + \sum_{i=1}^{n} A_{pt}\sigma_{pt}\right) - \left|\frac{\Delta A_{sc}}{A_{sc}} \times \sum_{i=1}^{n} A_{sc} f_s\right| = \frac{(M_{PE} + M_{CE} + k_{M3} \times M_{LL}^*)}{\Phi_b z}$$

**Location**

16.96 m from the Abutment 1 Centreline (for comparison to M1 check)

**Section Parameters**

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1439\ mm$ | $A_{sv} = 402\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1439\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 541\ mm$ | $\alpha_v = 90°$ | $A_o = 2.084 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 250\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.363 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 125\ mm$ | $A_{st.D/2} = 6132\ mm^2$ | $u_h = 6169\ mm$ |
| $d = 1543\ mm$ | $\sigma_{cp} = 6.157\ MPa$ | $A_{pt.D/2} = 13728\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 203.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1974 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt}\sigma_{pt} = 26480 \times 10^3 N$ |
| $k_{uo} = 0.125$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7$ $= 1281\ MPa$ | $\Phi M_u = 30492 \times 10^6\ Nmm$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25\Phi T_{CR} = 405.9 \times 10^6\ Nmm$ |

**Action Effects**

| | | |
|---|---|---|
| $M_{PE} = 11659 \times 10^6 Nmm$ | $V_{PE} = 92 \times 10^3 N$ | $T_{PE} = 239 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M_{LL}^* = 18320 \times 10^6 Nmm$ | $V_{LL}^* = 9 \times 10^3 N$ | $T_{LL}^* = 319 \times 10^6 Nmm$ |

**Solve for $k_{M3}$**

| | Case 1 | Case 2 | Case 3 | Case 4 |
|---|---|---|---|---|
| $k_{M3}$ solved using iteration | 1.03 | 1.03 | 1.03 | 1.03 |
| $\varepsilon_x$ | $0.474 \times 10^{-3}$ | $0.074 \times 10^{-3}$ | $0.559 \times 10^{-3}$ | $0.087 \times 10^{-3}$ |
| $\varepsilon_x$ sign check | Selected | Ignored | Selected | Ignored |
| $k_{M3}$ selected | 1.03 | | 1.03 | |
| $T^*$ | $567 \times 10^6$ N-mm | | $567 \times 10^6$ N-mm | |
| Check $T^* > 0.25\Phi T_{CR}$ | Ignored* | | Selected | |
| $k_{M3}$ | 1.03 or 103% | | | |

* If selected, the T* (calculated value for rating vehicle factored with selected $k_{M3}$) should be re-calculated as an uncracked sectional analysis and the re-check as per Eq. 8.2.1.2(1) of AS5100.5:2017. If torsion needs to be considered, ignore Case 1/2 and select Case 3/4.

## B.8 CALCULATE LOAD RATING FACTOR $k_{M4}$ FOR SCENARIO M4

**Scenario M4 (SLS Longitudinal Bending Load Rating)**

The Serviceability Limit State (SLS) Longitudinal bending load rating is calculated in accordance with AS 5100.5:2017 Clause 8.6.2.1(b) in which the increment in steel stress near the tension face is limited to the value obtained from Table 8.6.2.1 where the increment is defined as the increase from the value when the extreme concrete tensile fibre is at zero stress to the SLS load combination values. The moment on the cross section when the extreme concrete tensile fibre is at zero is defined as the decompression moment. The moment which corresponds to the achievement of the increment in the steel stress from the value determined at the decompression moment is defined as the serviceability bending capacity of the beam ($M_s$).

(Note: Detailed calculations for other provisions of AS 5100.5:2017 Cl. 8.6.2 are not shown in this example; but needs to be considered while calculating SLS crack control load rating.)

The load rating factor $k_{M4}$ for scenario M4 is the factor of live loads that results in full utilisation of the SLS bending capacity of the cross section under consideration. The objective function is,

$$M_{sls} = M_{PE,sls} + M_{CE,sls} + k_{M4} \times M^*_{LL,sls}$$

Where:

$M_{sls}$ = Serviceability capacity in bending (per beam)
$M_{PE,sls}$ = Serviceability design moment due to all Permanent Effects (PE)
$M_{CE,sls}$ = Serviceability design moment due to co-existing live load
$M_{LL,sls}$ = Factored serviceability design moment due to Vehicle Live Load effects (Including Gamma and DLA)

This equation needs to be solved for $k_{M4}$.

**Location**

16.96 m from the centreline of the abutment 1 bearing (approximately at the mid-span of span 1) with beam flange width 5056 mm.

**Action Effects and Capacity**

$M_{PE,sls} = 9342\ kNm$
$M_{CE,sls} = 0\ kNm$
$M_{LL,sls} = 10209\ kNm$
$M_s = 24172\ kNm$

**Solve for $k_{M4}$**

Solving for $k_{M4}$ we determine its value to be 1.453 or 145%.

## B.9 CALCULATE LOAD RATING FACTOR $k_{V1}$ FOR SCENARIO V1

**Scenario V1**

The load rating factor $k_{V1}$ for scenario V1 is the factor of live loads that results in the utilisation of the shear capacity. Main Roads WA SES Circular 01-20 amendment to shear design formulas amended the AS 5100.5:2017 Eq. 8.2.3.1 and the objective function is,

$$\Phi_s V_u = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

where,

$$\Phi_s V_u = \Phi_s V_{uc} + \Phi_s V_{us} + P_v \leq V_{u,max}$$

or,

$$\Phi_s V_u = \Phi_s k_v \sqrt{f_c'} b_v d_v + \frac{\Phi_s A_{sv} f_{sy} d_v}{s} (\sin(\alpha_v)\cot(\theta_v) + \cos(\alpha_v)) + P_v \leq \Phi_s V_{u,max}$$

and,

$$\sqrt{f_c'} \leq 8MPa$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

This equation needs to be solved for $k_{V1}$. As the value of $\theta_v$ in the objective function depends on the action effects, it is also dependent on the value $k_{V1}$. The value of $k_{V1}$, therefore, needs to be determined through using appropriate iterative solvers (for example using *goalseek* function). Furthermore, according to AS 5100.5:2017 Cl. 8.2.4, longitudinal strain $\varepsilon_x$ needs to be calculated based on one of the four alternative equations determined by their sign and magnitude of the torsional action effect $T^*$. But, the value of $T^*$ is also dependent on $k_{V1}$. Thus, four separates $k_{V1}$ needs to be calculated through iterative processes for following four alternative cases. The final value of $k_{V1}$, then, needs to be selected based on the sign of $\varepsilon_x$ and magnitude of $T^*$ compared against $0.25\Phi T_{CR}$. Please note, as per AS 5100.5:2017 Cl. 8.2.4.3 and 8.2.4.4, for sections closer than $d_o$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_o$ from the face of the support may be used in evaluating $\varepsilon_x$ and $k_v$.

***Case 1 – shear only and $\boldsymbol{\varepsilon_x \leq 3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V1} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V1} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2(E_sA_{st} + E_pA_{pt})}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$V_{u,max} = 0.55f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

$$\Phi_s V_u = \Phi_s k_v \sqrt{f_c'} b_v d_v + \frac{\Phi_s A_{sv} f_{sy} d_v}{s}(\sin(\alpha_v)\cot(\theta_v) + \cos(\alpha_v)) + P_v \leq \Phi_s V_{u,max}$$

$$\Phi_s V_u = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

***Case 2 – shear only and $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V1} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V1} * T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2(E_sA_{st} + E_pA_{pt} + E_cA_{ct})}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left( \frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)} \right) + P_v$$

$$\Phi_s V_u = \Phi_s k_v \sqrt{f_c'} b_v d_v + \frac{\Phi_s A_{sv} f_{sy} d_v}{s} (\sin(\alpha_v) \cot(\theta_v) + \cos(\alpha_v)) + P_v \leq \Phi_s V_{u,max}$$

$$\Phi_s V_u = V_{PE} + V_{CE} + k_{V1} * V_{LL}^*$$

***Case 3 – combined shear & torsion and*** $\boldsymbol{\varepsilon_x \leq 3.0 \times 10^{-3}}$

$$M^* = M_{PE} + M_{CE} + k_{V1} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V1} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt} f_{po}}{2(E_s A_{st} + E_p A_{pt})}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left( \frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)} \right) + P_v$$

$$\Phi_s V_u = \Phi_s k_v \sqrt{f_c'} b_v d_v + \frac{\Phi_s A_{sv} f_{sy} d_v}{s} (\sin(\alpha_v) \cot(\theta_v) + \cos(\alpha_v)) + P_v \leq \Phi_s V_{u,max}$$

$$\Phi_s V_u = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

***Case 4 – combined shear & torsion and*** $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$

$$M^* = M_{PE} + M_{CE} + k_{V1} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V1} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt} f_{po}}{2(E_s A_{st} + E_p A_{pt} + E_c A_{ct})}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left( \frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)} \right) + P_v$$

$$\Phi_s V_u = \Phi_s k_v \sqrt{f_c'} b_v d_v + \frac{\Phi_s A_{sv} f_{sy} d_v}{s} (\sin(\alpha_v)\cot(\theta_v) + \cos(\alpha_v)) + P_v \le \Phi_s V_{u,max}$$

$$\Phi_s V_u = V_{PE} + V_{CE} + k_{V1} \times V_{LL}^*$$

**Location**

8.68 m from the Abutment 1 Centreline.

**Section Parameters**

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1439\ mm$ | $A_{sv} = 402\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1439\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 541\ mm$ | $\alpha_v = 90°$ | $A_o = 2.084 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 250\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.363 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 125\ mm$ | $A_{st.D/2} = 6132\ mm^2$ | $u_h = 6169\ mm$ |
| $d = 1543\ mm$ | $\sigma_{cp} = 6.142\ MPa$ | $A_{pt.D/2} = 13728\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 203.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1974 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt} \sigma_{pt} = 26480 \times 10^3 N$ |
| $k_{uo} = 0.125$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7$ $= 1281\ MPa$ | $\Phi M_u = 30492 \times 10^6\ Nmm$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25 \Phi T_{CR} = 405.5 \times 10^6\ Nmm$ |

**Action Effects**

| | | |
|---|---|---|
| $M_{PE} = 9241 \times 10^6 Nmm$ | $V_{PE} = 683 \times 10^3 N$ | $T_{PE} = 348 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M_{LL}^* = 12510 \times 10^6 Nmm$ | $V_{LL}^* = 1463 \times 10^3 N$ | $T_{LL}^* = 458 \times 10^6 Nmm$ |

**Solve for $k_{V1}$**

| | Case 1 | Case 2 | Case 3 | Case 4 |
|---|---|---|---|---|
| $k_{V1}$ solved using iteration | 1.33 | 1.57 | 1.32 | 1.56 |
| $\varepsilon_x$ | 0.391 x$10^{-3}$ | 0.108 x$10^{-5}$ | 0.411 x$10^{-3}$ | 0.113 x$10^{-3}$ |
| $\varepsilon_x$ sign check | Selected | Ignored | Selected | Ignored |
| $k_{V1}$ selected | 1.33 | | 1.32 | |
| $T^*$ | 958x$10^6$ N-mm | | 952 x$10^6$ N-mm | |
| Check $T^* > 0.25\Phi T_{CR}$ | Ignored* | | Selected | |
| $k_{V1}$ | 1.32 or 132% | | | |

* If selected, the T* (calculated value for rating vehicle factored with selected $k_{V1}$) should be re-calculated as an uncracked sectional analysis and the re-check as per Eq. 8.2.1.2(1) of AS5100.5:2017. If torsion needs to be considered, ignore Case 1/2 and select Case 3/4.

## B.10 CALCULATE LOAD RATING FACTOR $k_{V2}$ FOR SCENARIO V2

**Scenario V2**

The load rating factor $k_{V2}$ for scenario V2 is the factor of live loads that results in the utilisation of the web crushing capacity. Main Roads WA SES Circular 01-20 amendment to shear design formulas amended the AS 5100.5:2017 Eq. 8.2.4.5 and the objective function is,

For box sections,

(i) Where wall thickness $t_w > A_{oh}/u_h$

$$\frac{\Phi_b V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*) * u_h}{1.7 A_{oh}^2}$$

(ii) Where wall thickness $t_w \leq A_{oh}/u_h$

$$\frac{\Phi_b V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*)}{1.7 t_w A_{oh}}$$

For other sections,

$$\frac{\Phi_b V_{u,max}}{b_v d_v} = \sqrt{\left[\frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v}\right]^2 + \left[\frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*) \times u_h}{1.7 A_{oh}^2}\right]^2}$$

where,

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

This equation needs to be solved for $k_{V2}$. As the value of $\theta_v$ in the objective function depends on the action effects, it is also dependent on the value $k_{V2}$. The value of $k_{V2}$, therefore, needs to be determined through using appropriate iterative solvers (for example using *goalseek* function). Furthermore, according to AS 5100.5:2017 Cl. 8.2.4, longitudinal strain $\varepsilon_x$ needs to be calculated based on one of the four alternative equations determined by their sign and magnitude of the torsional action effect $T^*$. But, the value of $T^*$ is also dependent on $k_{V2}$. Thus, four separates $k_{V2}$ needs to be calculated through iterative processes for following four alternative cases. The final value of $k_{V2}$, then, needs to be selected based on the sign of $\varepsilon_x$ and magnitude of $T^*$ compared against $0.25\Phi T_{CR}$. Please note, as per AS 5100.5:2017 Cl. 8.2.4.3 and 8.2.4.4, for sections closer than $d_o$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_o$ from the face of the support may be used in evaluating $\varepsilon_x$ and $k_v$.

***Case 1 – shear only and $\varepsilon_x \leq 3.0 \times 10^{-3}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

As $t_w \leq A_{oh}/u_h$,

$$\frac{\Phi_s V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*)}{1.7 t_w A_{oh}}$$

***Case 2 – shear only and $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

As $t_w \leq A_{oh}/u_h$,

$$\frac{\Phi_s V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*)}{1.7 t_w A_{oh}}$$

***Case 3 – combined shear & torsion and $\boldsymbol{\varepsilon_x \leq 3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt} f_{po}}{2\left(E_s A_{st} + E_p A_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

As $t_w \leq A_{oh}/u_h$,

$$\frac{\Phi_s V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*)}{1.7 t_w A_{oh}}$$

***Case 4 – combined shear & torsion and $\boldsymbol{\varepsilon_x \geq -0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt} f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$V_{u,max} = 0.55 f_c' b_v d_v \left(\frac{\cot(\theta_v)}{1 + \cot^2(\theta_v)}\right) + P_v$$

As $t_w \leq A_{oh}/u_h$,

$$\frac{\Phi_s V_{u,max}}{b_v d_v} = \frac{(V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*)}{b_v d_v} + \frac{(T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*)}{1.7 t_w A_{oh}}$$

**Location**

1.93 m from the Abutment 1 Centreline.

**Section Parameters**

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1347\ mm$ | $A_{sv} = 628\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1393\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 559\ mm$ | $\alpha_v = 90°$ | $A_o = 1.976 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 400\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.354 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 200\ mm$ | $A_{st.D/2} = 6584\ mm^2$ | $u_h = 6157\ mm$ |
| $d = 1548\ mm$ | $\sigma_{cp} = 4.011\ MPa$ | $A_{pt.d/2} = 8866\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 157.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1439.3 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt} \sigma_{pt} = 20402.3 \times 10^3 N$ |
| $k_{uo} = 0.097$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7 = 1281\ MPa$ | $\Phi M_u = N/A$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25\Phi T_{CR} = 532.2 \times 10^6\ Nmm$ |

**Action Effects**

| | | |
|---|---|---|
| $M_{PE} = 1933 \times 10^6 Nmm$ | $V_{PE} = 1370 \times 10^3 N$ | $T_{PE} = 378 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M_{LL}^* = 3318 \times 10^6 Nmm$ | $V_{LL}^* = 2331 \times 10^3 N$ | $T_{LL}^* = 480 \times 10^6 Nmm$ |

**Solve for $k_{V2}$**

| | Case 1 | Case 2 | Case 3 | Case 4 |
|---|---|---|---|---|
| $k_{V2}$ solved using *iteration* | 0.88 | 1.08 | 0.89 | 1.08 |
| $\varepsilon_x$ | $-0.734 \times 10^{-3}$ | $-0.066 \times 10^{-3}$ | $-0.694 \times 10^{-3}$ | $-0.062 \times 10^{-3}$ |
| $\varepsilon_x$ sign check | Ignored | Selected | Ignored | Selected |
| $k_{V2}$ selected | 1.08 | | 1.08 | |
| $T^*$ | $894 \times 10^6$ N-mm | | $895 \times 10^6$ N-mm | |
| Check $T^* > 0.25\Phi T_{CR}$ | Ignored* | | Selected | |
| $k_{V2}$ | 1.08 or 108% | | | |

* If selected, the T* (calculated value for rating vehicle factored with selected $k_{V2}$) should be re-calculated as an uncracked sectional analysis and the re-check as per Eq. 8.2.1.2(1) of AS5100.5:2017. If torsion needs to be considered, ignore Case 1/2 and select Case 3/4.

## B.11 CALCULATE LOAD RATING FACTOR $k_{V3}$ FOR SCENARIO V3 AND $k_{V4}$ FOR SCENARIOS V4

The load rating factor $k_{V3}$ should be calculated following the similar method for calculating $k_{M2}$ and $k_{V4}$ should be calculated following the similar method for calculating $k_{M3}$.

## B.12 CALCULATE LOAD RATING FACTOR $k_{V5}$ FOR SCENARIO V5

**Scenario V5**

The load rating factor $k_{V2}$ for scenario V2 is the factor of live loads that results in the utilisation of the closed tie cross sectional area for shear and coexisting torsion. The objective function is,

For closed tie reinforcement,

$$\frac{A_{sv}}{2} = \left[0.5 \times \left\{\frac{(V_{PE} + V_{CE} + k_{V5} \times V_{LL}^*) - \Phi_s V_{uc}}{\Phi_s f_{sy}\left(\frac{d_v}{s}\right)\cot(\theta_v)}\right\} + \frac{(T_{PE} + T_{CE} + k_{V5} \times T_{LL}^*) \times s}{\Phi_t 2A_o\, f_{sy}\cot(\theta_v)}\right]$$

where,

$A_{sv}$ = Total cross-sectional area of closed tie reinforcement provided to resist shear and torsion. Thus, $A_{sv}\,/2$ the cross-sectional area provided to resist shear and torsion for the critical web.

In principle, this equation states that the utilisation of the closed tie reinforcement occurs when the demand from shear on one web is added to the demand from co-existing torsion. Please note, the demand from torsion is additive to one web and deductive on the other. The V5 formula has been provided for the critical web where the effects are additive and assumes that the design shear is shared evenly between all webs in the cross section based on the webs having the save stiffnesses.

This equation needs to be solved for $k_{V5}$. As the value of $\theta_v$ in the objective function depends on the action effects, it is also dependent on the value $k_{V5}$. The value of $k_{V5}$, therefore, needs to be determined through using appropriate iterative solvers (for example using *goalseek* function). Furthermore, according to AS 5100.5:2017 Cl. 8.2.4, longitudinal strain $\varepsilon_x$ needs to be calculated based on one of the four alternative equations determined by their sign and magnitude of the torsional action effect $T^*$. But, the value of $T^*$ is also dependent on $k_{V5}$. Thus, four separates $k_{V5}$ needs to be calculated through iterative processes for following four alternative cases. The final value of $k_{V5}$, then, needs to be selected based on the sign of $\varepsilon_x$ and magnitude of $T^*$ compared against $0.25\Phi T_{CR}$. Please note, as per AS 5100.5:2017 Cl. 8.2.4.3 and 8.2.4.4, for sections closer than $d_o$ to the face of the support, the value of $\varepsilon_x$ calculated at $d_o$ from the face of the support may be used in evaluating $\varepsilon_x$ and $k_v$.

***Case 1 – shear only and $\varepsilon_x \leq 3.0 \times 10^{-3}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\Phi_s V_{uc} = \Phi_s k_v \sqrt{f_c'} b_v d_v$$

$$\frac{A_{sv}}{2} = \left[0.5 \times \left\{\frac{(V_{PE} + V_{CE} + k_{V5} \times V_{LL}^*) - \Phi_s V_{uc}}{\Phi_s f_{sy}\left(\frac{d_v}{s}\right)\cot(\theta_v)}\right\} + \frac{(T_{PE} + T_{CE} + k_{V5} \times T_{LL}^*) \times s}{\Phi_t 2A_o f_{sy}\cot(\theta_v)}\right]$$

Note: for calculating $k_v$, use limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 3.0 \times 10^{-3}$

***Case 2 – shear only and $\boldsymbol{\varepsilon_x} \geq \mathbf{-0.2 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\left|\frac{M^*}{d_v} + V^*\right| - P_v + 0.5N^* - A_{pt}f_{po}}{2\left(E_s A_{st} + E_p A_{pt} + E_c A_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\Phi_s V_{uc} = \Phi_s k_v \sqrt{f_c'} b_v d_v$$

$$\frac{A_{sv}}{2} = \left[0.5 \times \left\{\frac{(V_{PE} + V_{CE} + k_{V5} \times V_{LL}^*) - \Phi_s V_{uc}}{\Phi_s f_{sy}\left(\frac{d_v}{s}\right)\cot(\theta_v)}\right\} + \frac{(T_{PE} + T_{CE} + k_{V5} \times T_{LL}^*) \times s}{\Phi_t 2A_o f_{sy}\cot(\theta_v)}\right]$$

Note: for calculating $k_v$, use limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 3.0 \times 10^{-3}$

***Case 3 – combined shear & torsion and $\boldsymbol{\varepsilon_x} \leq \mathbf{3.0 \times 10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_sA_{st} + E_pA_{pt}\right)}$$

$$\theta_v = (29 + 7000 \times \varepsilon_x)$$

$$\Phi_s V_{uc} = \Phi_s k_v \sqrt{f_c'} b_v d_v$$

$$\frac{A_{sv}}{2} = \left[0.5 \times \left\{\frac{(V_{PE} + V_{CE} + k_{V5} \times V_{LL}^*) - \Phi_s V_{uc}}{\Phi_s f_{sy}\left(\frac{d_v}{s}\right)\cot(\theta_v)}\right\} + \frac{(T_{PE} + T_{CE} + k_{V5} \times T_{LL}^*) \times s}{\Phi_t 2A_o\, f_{sy}\cot(\theta_v)}\right]$$

Note: for calculating $k_v$, use limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 3.0 \times 10^{-3}$

***Case 4 – combined shear & torsion and $\boldsymbol{\varepsilon_x} \geq \mathbf{-0.2} \times \mathbf{10^{-3}}$***

$$M^* = M_{PE} + M_{CE} + k_{V2} \times M_{LL}^*$$

$$V^* = V_{PE} + V_{CE} + k_{V2} \times V_{LL}^*$$

$$T^* = T_{PE} + T_{CE} + k_{V2} \times T_{LL}^*$$

$$\varepsilon_x = \frac{\frac{M^*}{d_v} + \sqrt{(V^* - P_v)^2 + \left(\frac{0.9 \times T^* \times u_h}{2A_o}\right)^2} + 0.5N^* - A_{pt}f_{po}}{2\left(E_sA_{st} + E_pA_{pt} + E_cA_{ct}\right)}$$

$$\theta_v = 29 + 7000 \times \varepsilon_x$$

$$\Phi_s V_{uc} = \Phi_s k_v \sqrt{f_c'} b_v d_v$$

$$\frac{A_{sv}}{2} = \left[0.5 * \left\{\frac{(V_{PE} + V_{CE} + k_{V5} \times V_{LL}^*) - \Phi_s V_{uc}}{\Phi_s f_{sy}\left(\frac{d_v}{s}\right)\cot(\theta_v)}\right\} + \frac{(T_{PE} + T_{CE} + k_{V5} \times T_{LL}^*) \times s}{\Phi_t 2A_o\, f_{sy}\cot(\theta_v)}\right]$$

Note: for calculating $k_v$, use limits $-0.2 \times 10^{-3} \leq \varepsilon_x \leq 3.0 \times 10^{-3}$

**Location**

8.68 m from the Abutment 1 Centreline.

## Section Parameters

| | | | |
|---|---|---|---|
| $\alpha_2 = 0.85$ | $z = 1439\ mm$ | $A_{sv} = 402\ mm^2$ | $A_{cp} = 2.656 \times 10^6\ mm^2$ |
| $\gamma = 0.70$ | $d_v = 1439\ mm$ | $s = 150\ mm$ | $u_c = 6540\ mm$ |
| $\Phi_b = 0.80$ | $\bar{y} = 541\ mm$ | $\alpha_v = 90°$ | $A_o = 2.084 \times 10^6\ mm^2$ |
| $\Phi_s = 0.70$ | $b_v = 250\ mm$ | $A_{sc} = 6341\ mm^2$ | $A_{oh} = 2.363 \times 10^6\ mm^2$ |
| $\Phi_t = 0.70$ | $t_w = 125\ mm$ | $A_{st.D/2} = 6132\ mm^2$ | $u_h = 6169\ mm$ |
| $d = 1543\ mm$ | $\sigma_{cp} = 6.142\ MPa$ | $A_{pt.D/2} = 13728\ mm^2$ | $A_{ct} = 0.609 \times 10^6\ mm^2$ |
| $d_n = 203.6\ mm$ | $d_g = 20\ mm$ | $\Sigma A_{sc} f_s = -1974 \times 10^3 N$ | $\Sigma A_{st} f_s + \Sigma A_{pt} \sigma_{pt} = 26480 \times 10^3 N$ |
| $k_{uo} = 0.125$ | $P_v = 0.0N$ | $f_{po} = 1830 \times 0.7 = 1281\ MPa$ | $\Phi M_u = 30492 \times 10^6\ Nmm$ |
| $D = 1700\ mm$ | $\gamma_p = 0.90$ | | $0.25\Phi T_{CR} = 405.5 \times 10^6\ Nmm$ |

## Action Effects

| | | |
|---|---|---|
| $M_{PE} = 9241 \times 10^6 Nmm$ | $V_{PE} = 683 \times 10^3 N$ | $T_{PE} = 348 \times 10^6 Nmm$ |
| $M_{CE} = 0\ Nmm$ | $V_{CE} = 0\ N$ | $T_{CE} = 0\ Nmm$ |
| $M_{LL}^* = 12510 \times 10^6 Nmm$ | $V_{LL}^* = 1463 \times 10^3 N$ | $T_{LL}^* = 458 \times 10^6 Nmm$ |

## Solve for $k_{V5}$

| | Case 1 | Case 2 | Case 3 | Case 4 |
|---|---|---|---|---|
| $k_{V5}$ solved using *iteration* | 1.13 | 1.22 | 1.12 | 1.21 |
| $\varepsilon_x$ | 0.132x10$^{-3}$ | 0.037 x10$^{-3}$ | 0.149 x10$^{-3}$ | 0.042 x10$^{-3}$ |
| $\varepsilon_x$ sign check | Selected | Ignored | Selected | Ignored |
| $k_{V5}$ selected | 1.13 | | 1.12 | |
| $T^*$ | 867x10$^6$ N-mm | | 861x10$^6$ N-mm | |
| Check $T^* > 0.25\Phi T_{CR}$ | Ignored* | | Selected | |
| $k_{V5}$ | 1.12 or 112% | | | |

* If selected, the T* (calculated value for rating vehicle factored with selected $k_{V5}$) should be re-calculated as an uncracked sectional analysis and the re-check as per Eq. 8.2.1.2(1) of AS5100.5:2017. If torsion needs to be considered, ignore Case 1/2 and select Case 3/4.

# APPENDIX C

## Example of Load Rating Memo and Report (Non-Timber Bridges)

# APPENDIX C - EXAMPLE OF LOAD RATING MEMO AND REPORT

## C.1 MEMORANDUM

| | | |
|---|---|---|
| File | : | **20/9842** |
| To | : | **Engineer Bridge Loading** |
| Subject | : | **Bridge No. 1694 on Murdoch Drive over Farrington Road**<br>**Heavy Loads Assessment / Load Rating** |

1. In response to your request to assess the above bridge, a load rating check for SM1600, T44, HLP, Group 1 and Group 2 vehicles has been completed. A detailed Load Rating Report is attached with this memo.

2. The bridge is a two-span simply supported precast tee-roff structure on a skew (21.7°) and curve of approximately 477 m radius. The spans are 31.9 m and 21.6 m respectively and the 4 No. beams are spaced at 4.949 m, 5.302 m and 5.231 m, with an 18.5 m width between edge kerbs. A 2.5 m median kerb separates dual carriageways.

   At the abutments, each beam is supported on a column behind MSE wall panels, which are founded on spread footings. At the pier, 2 No. columns support the superstructure via a footing and piles. Elastomeric bearings are used at each end of each beam at both abutments and pier.

   The structure was built in 2019 and is owned and operated by Main Roads WA. Refer Load Rating Report for general arrangement of Bridge No. 1694.

3. The analysis incorporates the following approach:

   - 5 No. standard design lanes for SM1600, and 6 No. standard design lanes for T44 and Group 1 vehicles without the median kerb.
   - 2 No. lane carriageways in both directions, assessed with and without the median kerb for HLP and Group 2 vehicles.
   - All materials as per IFC drawings ("as new" condition).
   - Yield stress of 500 MPa in the reinforcement and 1500 MPa in the prestressed strands.
   - Precast concrete tee-roff beams grade 50 MPa and in-situ deck slab 40 MPa.
   - Moment re-distribution has not been considered in this load rating (structure is simply supported).
   - Moment and shear capacities were derived using modified compression field theory (MCFT).
   - Settlement and temperature were not considered for the simply supported structure.

4. Cross-sections have been checked at 0.1 m intervals along the length of the beam to account for the interaction between moment, shear and torsion and the cross-section capacities of the associated section.

5. The bridge was analysed using Structural Bridge Design (SBD) software. The grillage type, aspect ratio, node locations and transverse member orientation were determined in line with Bridge Deck Behaviour (Hambly, 1991).

Each beam was modelled by a longitudinal member using properties of the transformed beam-slab section, accounting for varying flange widths. The ends of beams were free to rotate over the pier, to model cracking expected in the link slabs.

Transverse members comprised rigid links extending to the webs of beams, and deck-slab members connected the rigid links of adjacent beams.

Refer Load Rating Report for grillage setup and models.

6. Torsional stiffness of all members was factored by 0.2 to account for cracking at ULS.

7. The section capacities were obtained using in-house software/spreadsheet. The minimum flange width of each span was used, and the deck-slab width was transformed using concrete section properties.

8. Dead loads included self-weight of the beams with 200 mm nominated minimum slab thickness plus an additional 50 mm allowance for beam hog.

   Guardrails, kerbs and 100 mm thick asphalt were applied as superimposed dead loads, factored by 1.4 as they are controlled by the relevant authority.

9. Vehicles were assessed to travel in both directions. Group 2 Vehicle 8 was limited in the 8.0 m wide carriageway to within 0.2 m of the kerbs. HLP and Group 2 vehicles were assessed with and without:

   - The 2.5 m median kerb; and
   - Co-existing effects (CE) of 50% SM1600 in the opposite carriageway.

10. External beams governed the load ratings for both spans. Bending and shear load rating factors were calculated for 3 No. cases; (a) Maximum moment, associated shear and torsion, (b) Maximum shear, associated moment and torsion and (c) Maximum torsion, associated moment and shear.

11. 4 No. bending ratings (M1-M4) and 5 No. shear ratings (V1-V5) were obtained to determine the controlling condition, as the interaction between bending, shear and torsion impacts the cross-section capacities and the critical locations also vary.

    Moment capacities were adjusted by percentage demand for bending, as a proportion of applied bending, shear and torsion.

    Shear capacities were adjusted by percentage demand for shear, as proportion of applied shear and torsion.

12. Based on the above criteria, the table below shows the summary of governing load ratings for each vehicle. s

| Load Rating Summary | | | | | |
|---|---|---|---|---|---|
| Vehicle | Span | Load Rating (%) | Location from Support CL (m)* | Governing Case | Governing Capacity |
| SM1600 | 1 | 98% | 16.957 | M | $k_{M2}$ |
| T44 | 2 | 157% | 10.297 | M | $k_{M1}$ |
| HLP 320 | 1 | 104% | 14.962 | M | $k_{M2}$ |
| HLP 400 | 1 | 89% | 16.159 | M | $k_{M2}$ |
| GROUP 1, VEHICLE 1 | 2 | 285% | 9.097 | M | $k_{M1}$ |
| GROUP 1, VEHICLE 2 | 1 | 208% | 14.962 | M | $k_{M2}$ |
| GROUP 1, VEHICLE 3 | 1 | 188% | 14.962 | M | $k_{M2}$ |
| GROUP 1, VEHICLE 4 | 1 | 194% | 14.962 | M | $k_{M2}$ |
| GROUP 2, VEHICLE 1 - 3.01 | 2 | 272% | 8.997 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 1 - 3.70 | 2 | 272% | 8.997 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 2 - 3.01 | 2 | 239% | 13.295 | V | $k_{V5}$ |
| GROUP 2, VEHICLE 2 - 3.70 | 2 | 250% | 9.097 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 4 | 2 | 174% | 10.297 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 4 - NS | 2 | 147% | 10.297 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 5 | 2 | 133% | 10.297 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 5 - NS | 2 | 112% | 10.297 | M | $k_{M1}$ |
| GROUP 2, VEHICLE 7 | 1 | 88% | 16.159 | M | $k_{M2}$ |
| GROUP 2, VEHICLE 8 | 1 | 70% | 16.159 | M | $k_{M2}$ |

Notes: $k_{m1}$ indicates no additional longitudinal force acquired from shear and torsion.
$\sigma_{pu}$ has been used as per AS 5100.5:2017 Clause 8.1.7.
*Support at end of lowest chainage.

13. The following have been checked and found not to govern the load ratings:

- Transverse bending and shear of the deck slab;
- Interface shear – longitudinal and transverse;
- Bearings, substructure and foundations; and
- Compression fan regions.

14. For your information, as requested.

ENGINEER
01 June 2022

## C.2 LOAD RATING REPORT

### C.2.1 Introduction

This load rating report of Bridge No. 1694 incorporates the shear strength analysis using the Modified Compression Field Theory (MCFT) approach of concrete sections in AS 5100.5:2017 (incorporating Amendment 1 issued 2018). It is similar to design guides AASHTO Load and Resistance Factor Design (LRFD) and Canadian Highway Bridge Design Code (CHBDC). Shear capacities were derived through combining design shear, bending and torsion actions at a given cross-section.

### C.2.2 List of Symbols and Abbreviations

| | |
|---|---|
| CE | Co-existing Loads |
| DL | Dead Loads |
| EBL | Engineer Bridge Loading |
| EQ | Earthquake (Load) |
| LL | Live Loads |
| MCFT | Modified Compression Field Theory |
| MRWA | Main Roads Western Australia |
| PE | Permanent Effects |
| PSP | Principal Shared Path |
| SDL | Superimposed Dead Loads |
| SES | Senior Engineer Structures |
| SLS | Serviceability Limit State |
| ULS | Ultimate Limit State |
| $d_o$ | Distance from the outermost compression fibre to the centroid of outer-most tensile reinforcement |
| $f'_C$ | Characteristic compressive (cylinder) strength of concrete at 28 days |
| $f_{PB}$ | Characteristic minimum breaking strength of tendons |
| $f_{PY}$ | Yield strength of tendons |
| $f_{PO}$ | Stress in prestressed reinforcement when stress in the surrounding concrete is zero |
| $f_{SY}$ | Characteristic yield strength of reinforcement |
| $A_P$ | Area of tendons |
| $E_C$ | Mean value of the modulus of elasticity of concrete at 28 days |
| $E_P$ | Modulus of elasticity of tendons |
| $E_S$ | Modulus of elasticity of reinforcement |
| $\theta_v$ | Angle between axis of the concrete compression strut and longitudinal axis of a member |
| M* | Design bending moment at a cross-section |
| $V^*_{LL}$ | Design shear force at a cross-section due to live loads |
| $V_{PE+CE}$ | Design shear force at a cross-section due to permanent effects and co-existing loads |
| $V_{SDL+CE}$ | Design shear force at a cross-section due to superimposed dead loads and co-existing loads |

| | |
|---|---|
| $V_{CORR}$ | Corresponding design shear force at a cross-section |
| $M_S$ | Design serviceability strength in bending at a cross-section, derived from maximum allowable increment of steel stress from the decompression moment |
| $\varphi M_U$ | Design ultimate strength in bending at a cross-section |
| $\varphi M_{U,M}$ | Design ultimate strength in bending at a cross-section adjusted by % tensile force demand from bending as a proportion of axial, bending, shear and torsion actions |
| $\varphi V_{UC}$ | Design ultimate shear strength at a cross-section excluding shear reinforcement |
| $q_{ALLOWABLE}$ | Design shear flow capacity at a cross-section |
| $q_{LL}$ | Shear flow at a cross-section due to live loads |
| $q_{PE+CE}$ | Shear flow at a cross-section due to permanent effects and co-existing loads |
| $q_{SDL+CE}$ | Shear flow at a cross-section due to superimposed dead loads and co-existing loads |

### C.2.3 References


Hambly, E. C. (1991). *Bridge deck behaviour*. CRC Press.

BBR Australia (1973). *BBR Multi-wire System: Prestressing Manual (metric).* BBR Australia Pty Ltd.

### C.2.4 Bridge Details

Bridge No. 1694 was constructed in 2019. It carries Murdoch Drive Extension over Farrington Road in the City of Cockburn, within the Metropolitan region. The bridge is owned and operated by Main Roads Western Australia (MRWA). Refer Figure C.1 for the location of the bridge.

The bridge is a two-span simply supported precast tee-roff structure on a skew (21.7°) and curve of approximately 477 m radius. The span lengths are 31.9 m and 20.6 m respectively and the 4No. beams are spaced at 4.949 m, 5.302 m and 5.231 m, with an 18.5 m width between kerbs. At the abutments, each beam is supported on a column located behind MSE walls, which are founded on a shared spread footing. At the Pier, 2No. blade piers support the superstructure via a shared footing and piles. Elastomeric bearings are used at both the abutments and the pier.

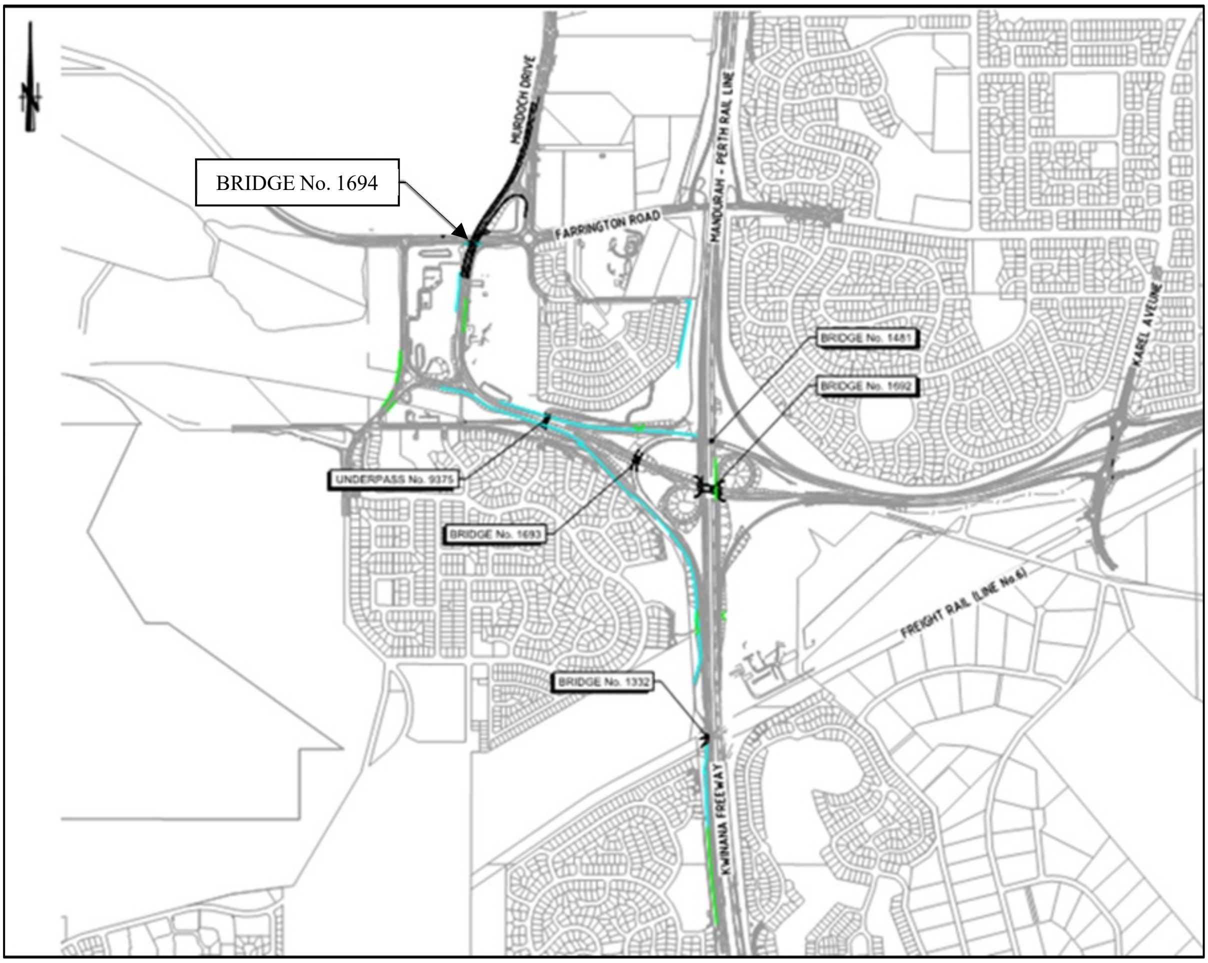



Figure C.1 Location of Bridge No. 1694

Bridge No. 1694 has dual-lane carriageways in both directions (north and south) separated by a 2.5 m wide median kerb. Figure C.2 and Figure C.3 show the plan and typical cross section of the bridge.

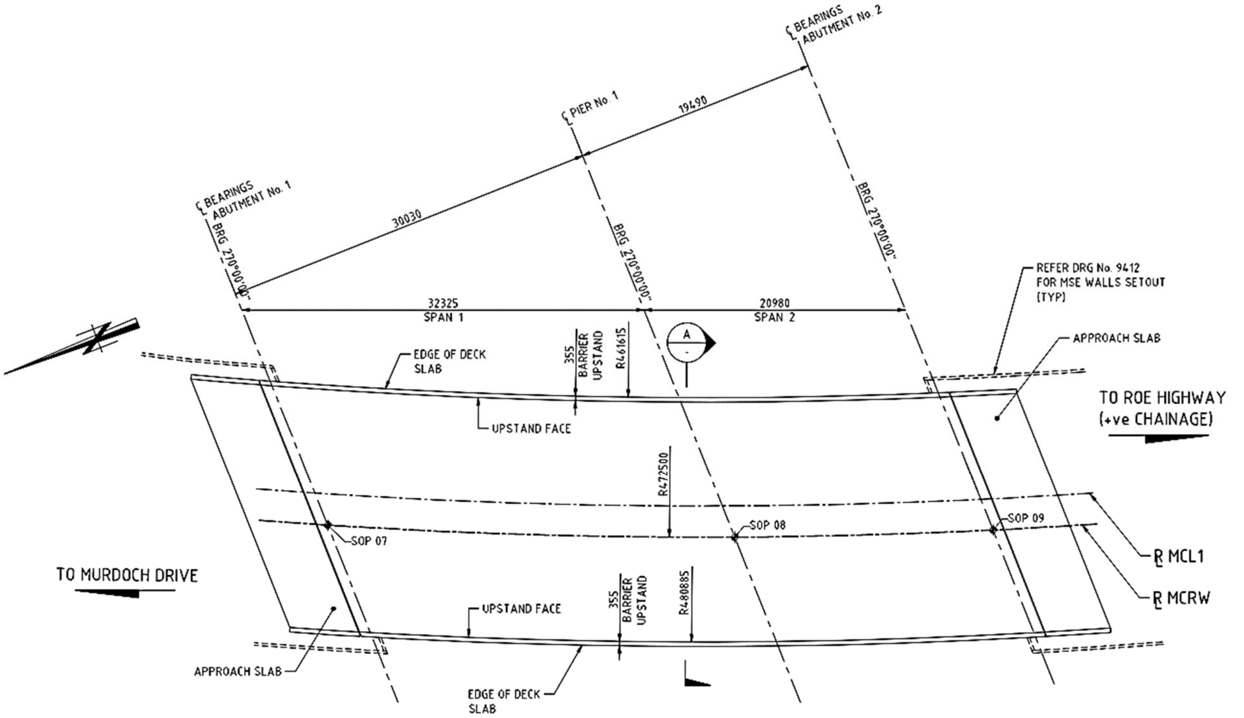



Figure C.2 Plan View of the Bridge

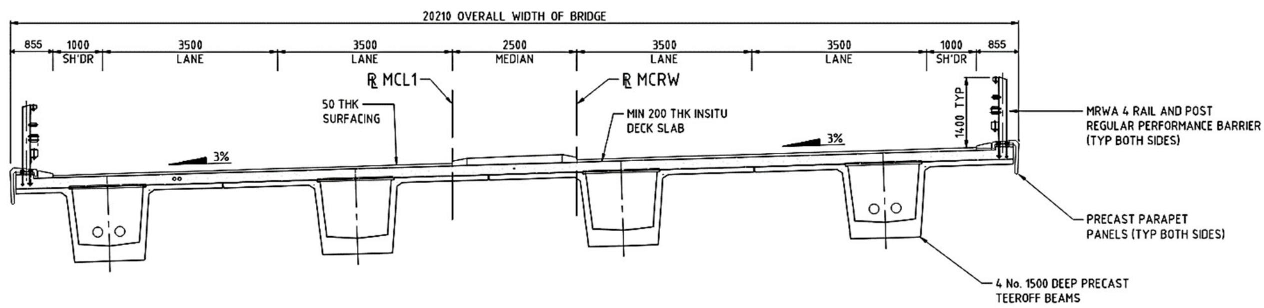



Figure C.3 Typical Cross Section of the Bridge

### C.2.5 Material Properties

Material properties of Bridge No. 1694 are shown in Table C.1. The bridge is approximately two years old and is in good condition.

Table C.1 Bridge No. 1694 Material Properties

| Material | Property | Symbol | Unit | Value | Reference |
|---|---|---|---|---|---|
| Prestress (15.2mm Dia. 7-Wire Super Strand) | Breaking Stress | $f_{PB}$ | MPa | 1830 | T 3.3.1 AS 5100.5:2017 |
| | Yield Stress | $f_{PY}$ | MPa | 1501 | Cl 3.3.1(b) AS 5100.5:2017 |
| | Elastic Modulus | $E_P$ | MPa | 195000 | Cl 3.3.2 AS 5100.5:2017 |
| | Area per Tendon | $A_P$ | $mm^2$ | 143 | T 3.3.1 AS 5100.5:2017 |
| Reinforcement | Yield Strength | $f_{SY}$ | MPa | 500 | T 3.2.1 AS 5100.5:2017 |
| | Elastic Modulus | $E_S$ | MPa | 200000 | Cl 3.2.2 AS 5100.5:2017 |
| Concrete Precast Beam | 28 Day Strength | $f'_C$ | MPa | 50 | 10-0107-040-BR-DG-9402 |
| | Elastic Modulus | $E_C$ | MPa | 34800 | T 3.1.2 AS 5100.5:2017 |
| Concrete In-situ Slab | 28 Day Strength | $f'_C$ | MPa | 40 | 10-0107-040-BR-DG-9402 |
| | Elastic Modulus | $E_C$ | MPa | 32800 | T 3.1.2 AS 5100.5:2017 |

### C.2.6 Critical Locations and Section Parameters

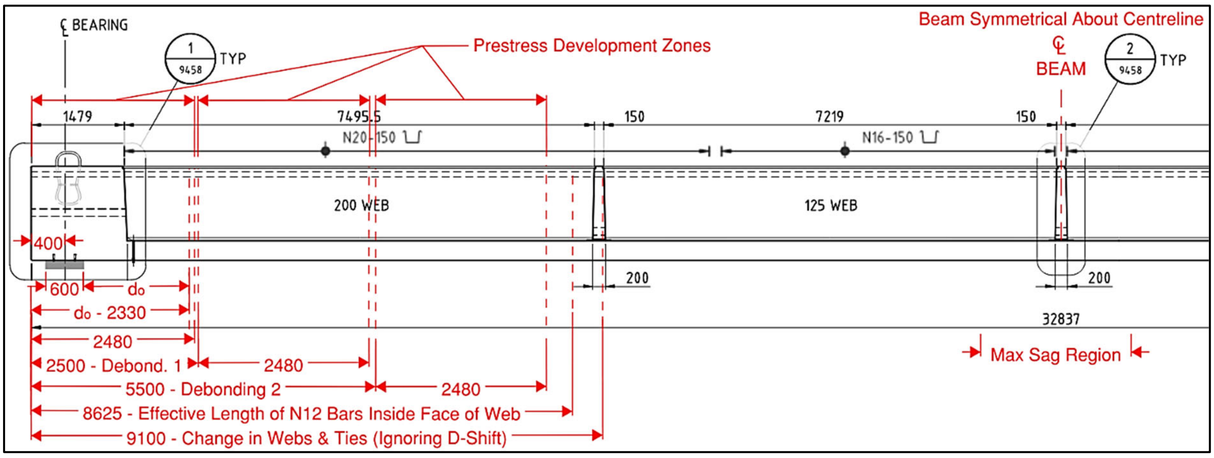
The beams are supported on bearings via a heavily reinforced diaphragm. A compression fan region (refer AS 5100.5:2017 Cl 8.2.9.2) was therefore idealised over supports and extended to $d_o$. The heavily reinforced end-diaphragms provided adequate stress paths through strut-and-tie action. The longitudinal load rating was therefore carried out starting from $d_o$ from face of bearings, at 0.1 m increments along the spans for all beams (complying with AS 5100.5:2017 Cl 8.2.9.1 requirements through detailed calculations rather than considering these requirements may be satisfied by extending the flexural tension reinforcements and tendons), to capture locations governed either by a change in section or a worst-case combination of shear, bending and torsion.

Conventional reinforcement was ignored within development zones. Development lengths of deformed bars at the ends of the beams were within $d_o$ from the face of support. While the development length of the internal web face bars was calculated to be 348 mm and in a non-critical location.

Prestress losses were calculated in accordance with Clause 3.3.4 and Clause 3.4 of

AS 5100.5:2017. Effective prestress within development zones was determined using a linear approach that started at zero at the end of debonding to fully developed at a distance in accordance with Equation 13.3.2.2 of AS 5100.5:2017. Development zones of de-bonded strands were checked for any concrete tension from SLS loading (2Lp check) in accordance with Clause 13.3.2.2. The corresponding effective prestress was used at each increment to assess capacities against design actions.
The critical locations along the beams marked by changes in cross-sectional properties are shown in Figure C.4 below.

<u>SPAN 1</u>




<u>SPAN 2</u>

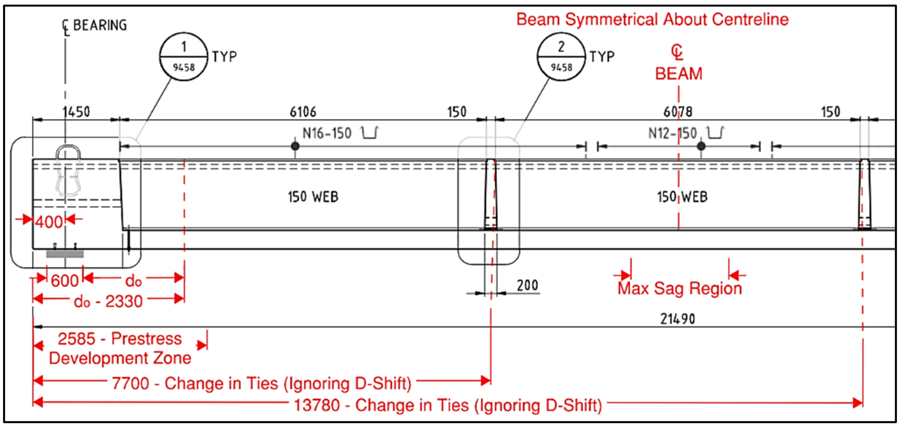



Figure C.4 Changes in Beam Section Properties (Some of the Critical Locations)

The prestress parameter $f_{po}$ outside the development length was taken as $0.7f_{pb}$ when calculating $\varphi V_{UC}$, as per notes in Clause 8.2.4 of AS 5100.5:2017. Capacities were derived using the minimum flange width in each span, with the deck-slab portion transformed to effective width to account for the difference in concrete properties between the precast beam and the in-situ slab. In the derivation of the ULS bending capacity ($\varphi M_U$) the 28-day concrete strength ratio was used, whilst for the SLS bending capacity ($M_S$) the ratio of the elastic moduli was used to determine the transformed deck-slab width. The difference in the methods accounts for concrete compression stress block approach for ULS, versus the linear stress-strain model for SLS.

### C.2.7 Modelling and Assumptions

AutoCAD was used to draw the curves and skews of Bridge No. 1694 and the grillage was constructed using Structural Bridge Design (SBD) software. An ACES line-beam was used to check the sum of longitudinal member design actions.
The grillage type, aspect ratio, node locations and transverse member orientation were determined in line with Bridge Deck Behaviour (Hambly, 1991).

**Grillage Details**

Each beam was modelled by a longitudinal member using properties of the transformed beam-slab section. Transverse members were perpendicular to longitudinal members, to match the orientation of deck reinforcement. Additional nodes were placed at the mid-point of transverse slab members to better capture wheel loads and provide preliminary transverse member design action outputs. Refer Figure C.5 and Figure C.6 for grillage setup and plan view.

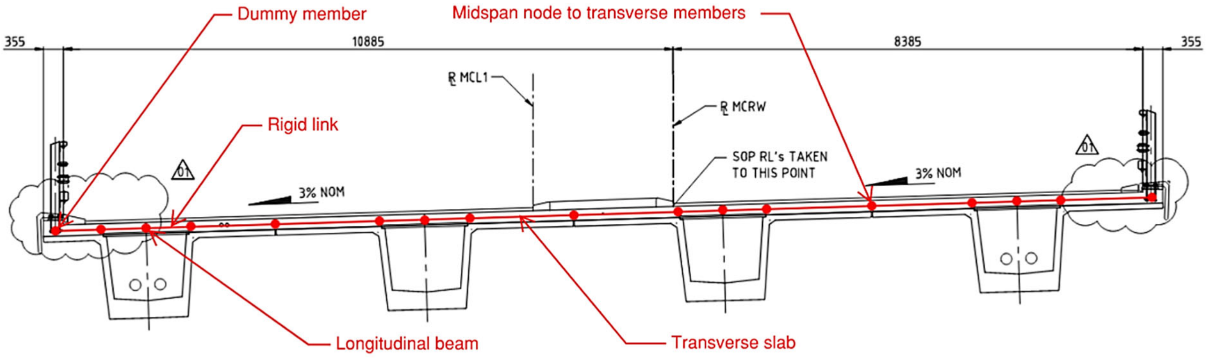
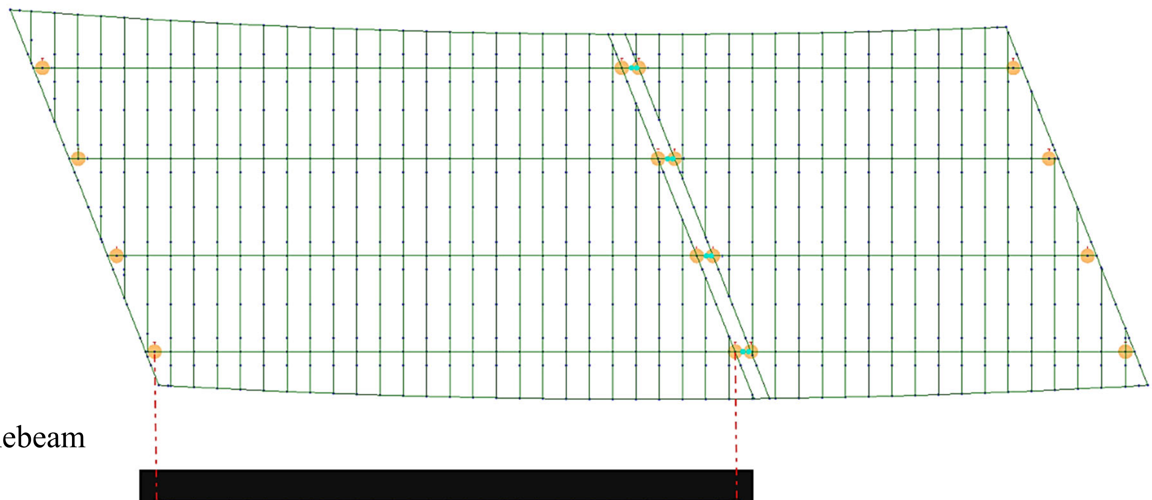



Figure C.5 2D Grillage Setup

<u>Grillage</u>



Linebeam

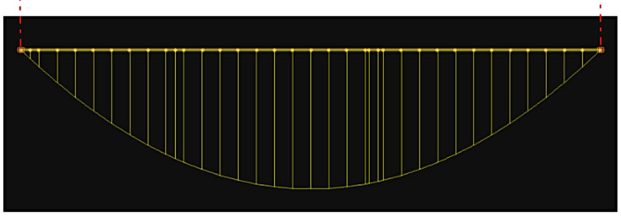


Figure C.6 Bridge Grillage and Line Beam Models

Longitudinal edge dummy members were coincident with the edge of the external beams, which was an approximate centroid to the combined upstand, barrier and overhanging fascia panels. Rigid links were at right angles to the longitudinal beams, extending to the web centrelines at the height of the centroid of the composite section. Transverse deck-slab members connected rigid links of adjacent tee-roff beams.

Longitudinal member ends were released over the piers to allow rotation of the beams, modelling a cracked link slab.

For transverse slab analysis, 3 No. evenly spaced longitudinal dummy members were introduced between tee-roffs. The additional members, and subsequent nodes, provided better local distribution and 'smoother' design action diagrams.

**Member Properties**

The longitudinal member properties of the Bridge No. 1694 tee-roffs were calculated for each grillage element. The torsional stiffness was calculated as per the method outlined in Bridge Deck Behaviour (Hambly, 1991). The torsional stiffness of all members was further factored by 0.2 as per AS 5100.5:2017 and with reference to Structures Engineering Design Manual (3912/03). The reduced torsional stiffness was applied for ULS analysis, as the torsional cracking moment capacity was exceeded.

Transverse slab members adopted the properties of the deck slab alone for conservative longitudinal analysis. Transverse assessment adopted the properties of 200 mm deck slab for mid-span bending and 300 mm combined deck and flange for bending and shear elsewhere.

Dummy members were assigned a low nominal stiffness to prevent computational error. Infinitely rigid pinned supports were conservatively used in the load rating assessment.

**Permanent Effects**

Bridge No. 1694 permanent effects considered only DL and SDL, without temperature and differential settlement, due to the simply supported spans.

Dead loads included self-weight of the tee-roff beams with the minimum 200 mm slab thickness plus an additional 50 mm allowance throughout for beam hog, which were multiplied by a factor of 1.2 for ULS in accordance with Table 6.2 in AS 5100.2:2017.

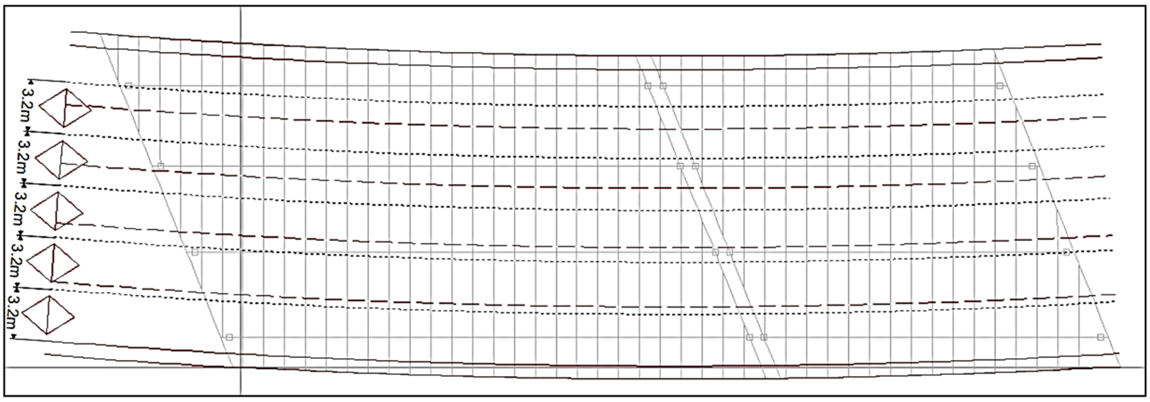
SDLs included 100 mm thick asphalt between kerbs, kerbs and guardrails, and were multiplied by a factor of 1.4 for ULS in accordance with Table 6.3 Item (b) in AS 5100.2:2017, as these are controlled by the relevant authority.

**Vehicles**

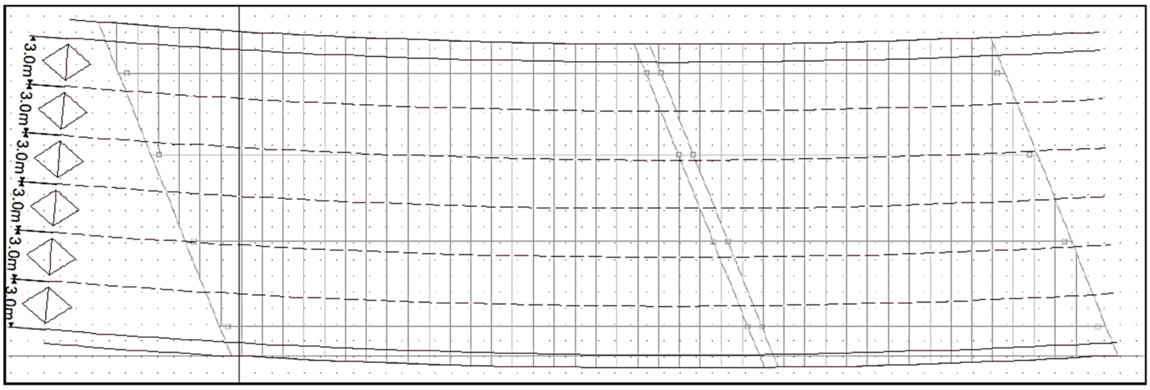
Bridge No. 1694 load rating included SM1600, T44, HLP, Group 1 and Group 2 vehicles. Design lanes were offset by the edge-kerb width for SM1600, T44 and Group 1 vehicles. HLP and Group 2 vehicle lanes were positioned considering movements with and without (a) the 2.5m median kerb; and (b) co-existing effects (CE) of 50% SM1600 in the opposite carriageway.

The movement of Group 2 Vehicle 8 was limited in the 8.0 m wide carriageway to within 0.2 m of the kerbs. Refer Figure C.7 for design lanes positions in the SBD model. All vehicles were assessed to travel in both directions.

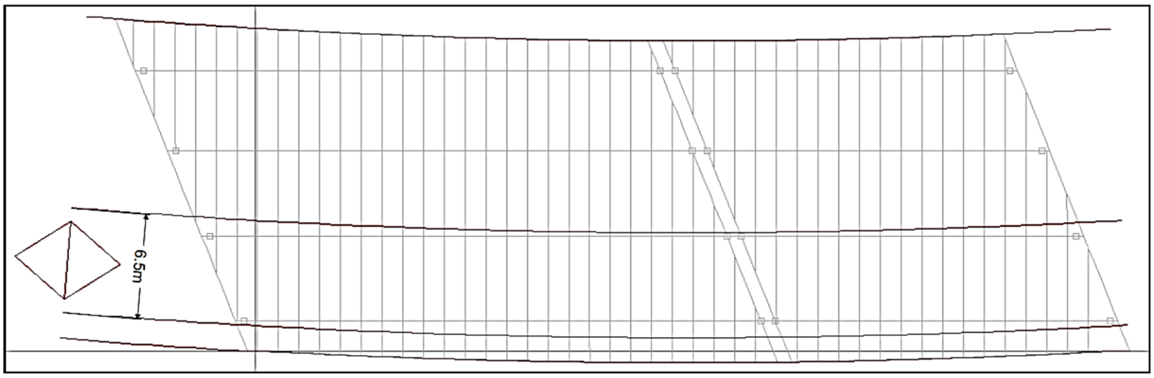
SM1600 Design Lanes



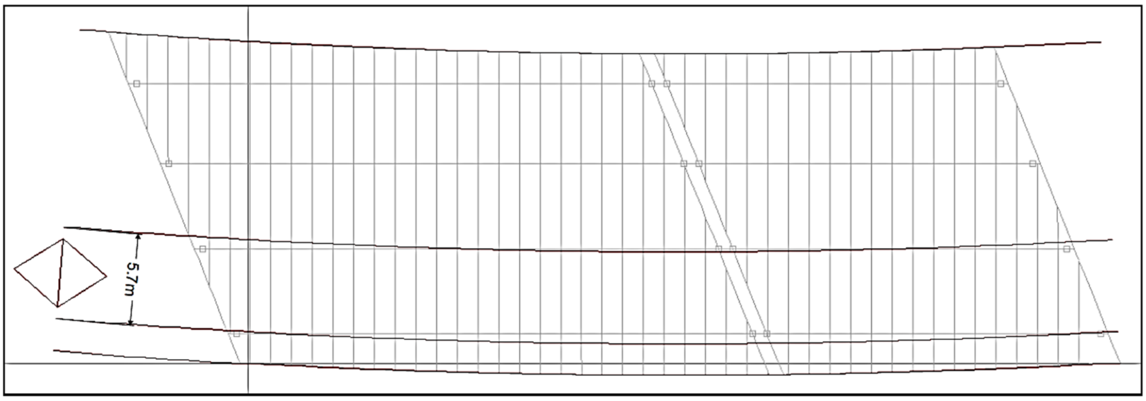

T44 Design Lanes



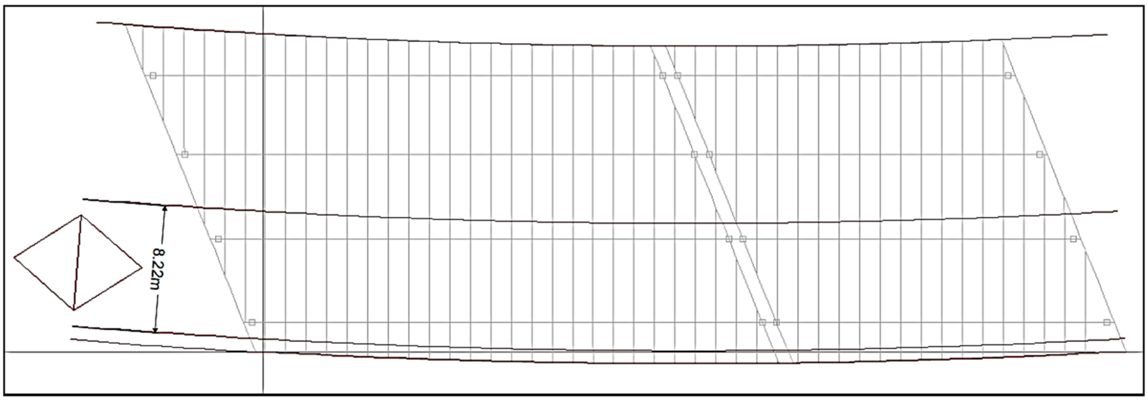

HLP400 Movement




Group 2 – 3.7 m Movement




Group 2 Vehicle 8 Movement




50% SM1600 Lanes in Opposite Carriageway

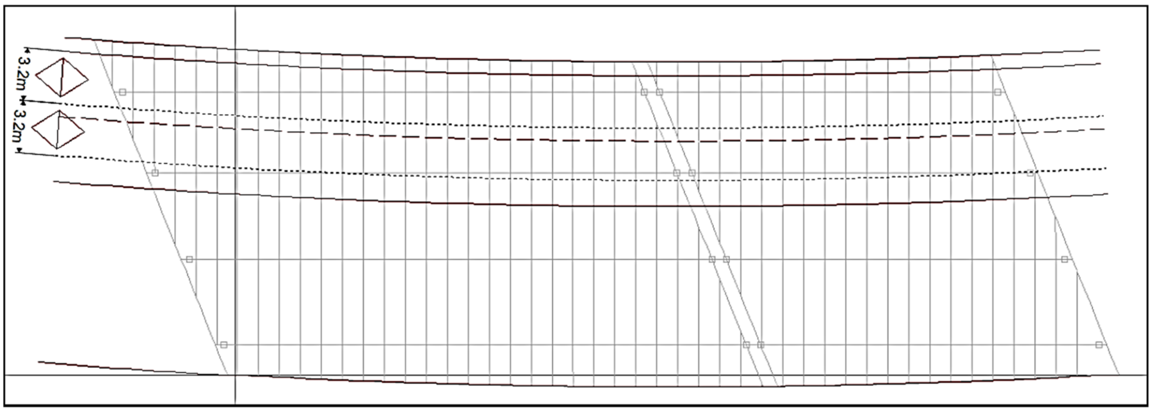



Figure C.7 Some of the Vehicle Lanes

**Analysis**

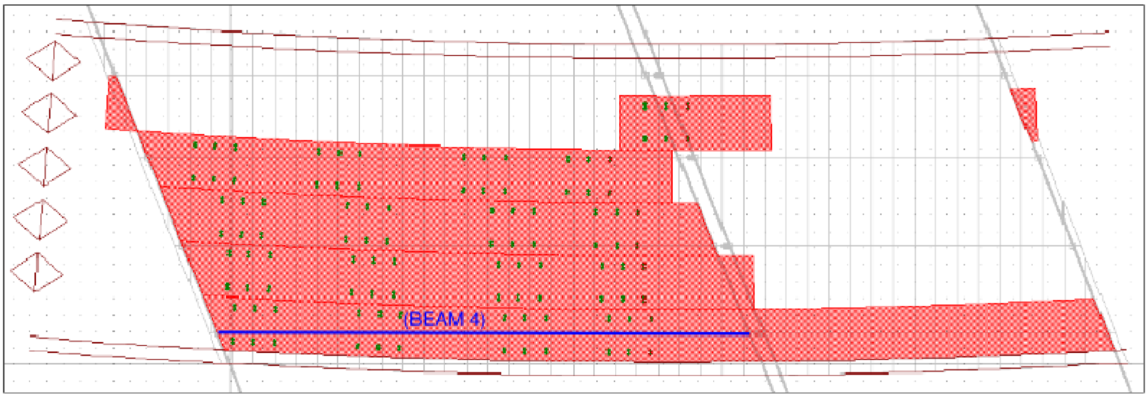
Longitudinal movement of the vehicles on Bridge No. 1694 grillage was refined to 0.1 m increments. Transverse movements were also refined to 0.1 m increments for HLP and Group 2 vehicles within the ±1.0 m offset from centreline of carriageway.

The critical number of lanes was derived separately for SM1600, T44 and Group 1 vehicles for each of the 3No. cases:

- Maximum Bending (+ve) – simply supported spans
- Maximum Shear (+/-ve)
- Maximum Torsion (+/-ve)

Refer Figure C.8 for governing number of lanes for SM1600 critical bending and shear cases.

SM1600 – Span 1 Sag (External Beam)




SM1600 – Span 1 Shear AT 8.68m from Abutment 1 Centreline (External Beam)

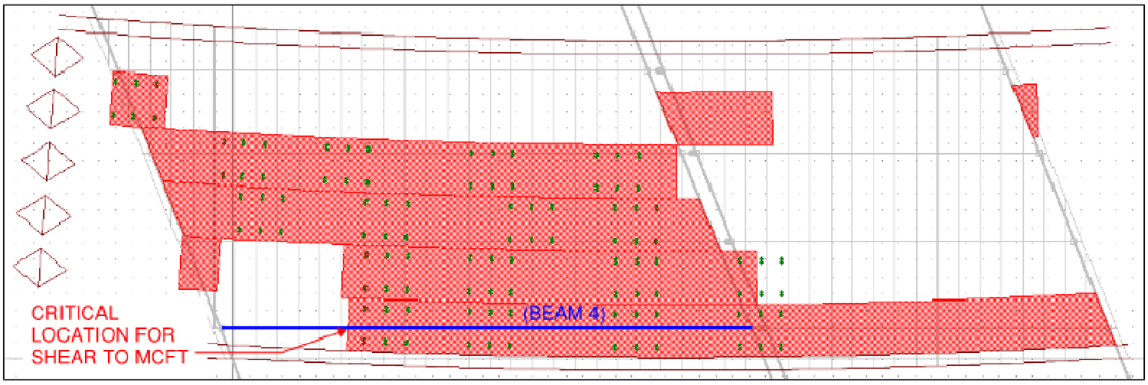



Figure C.8 SM1600 Patterned Loading for Critical Bending and Shear of Beam 4

**Grillage Results**

Positive and negative shear and torsion envelopes accounted for worst-case results at both ends of the beam. The direction of shear and torsion were recorded prior to combining with PE and CE effects. The maximum torsion case did not govern for any vehicles for Bridge No. 1694.

The output results of the critical combinations were visually cross-checked against diagrammatic grillage results in SBD. HLP and the longer Group 2 vehicles were checked by manual adjustment for critical loading on the curve.

Retaining the 2.5 m wide median kerb for HLP and Group 2 vehicles resulted in the highest design actions on the external beams. The median kerb offset the CE loading in the opposite carriageway further away from the critical edge beam in both spans, but still had a minor impact on the critical Span 1 edge beam. While the higher stiffness of shorter Span 2 prevented CE loads from reaching the external beam on the opposite side of the bridge.

**Output Data**

Beam PE data, as well as LL and CE data for the 3 No. cases (maximum bending, maximum shear and maximum torsion) were output with corresponding design actions. Shear values were averaged, and all output data was refined to 0.1 m increments using linear interpolation.

### C.2.8 Load Rating Results

The load rating was checked for each 0.1 m increment using in-house software, for each of the 3No. cases (maximum bending, maximum shear and maximum torsion) and for each of the checks M1-M4 and V1-V5. The critical section for each vehicle was checked with an Excel spreadsheet to confirm the ratings for both bending and shear.

**Longitudinal Actions**

The longitudinal bending and longitudinal shear load rating factors for Span 1 are shown in Table C.2 and for Span 2 are shown in Table C.3.

Table C.2 Bridge 1694 Span 1 Load Rating Factors (Longitudinal)

| Load Rating Factors - Span 1 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| SM1600 | M | 1.03 | **0.98** | 1.03 | 1.45 | 1.44 | 1.21 | NA | NA | 1.16 |
| | | 17.46 | 16.96 | 17.46 | 17.46 | 8.68 | 2.19 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.32 | **1.08** | 1.30 | 1.47 | 1.12 |
| | | NA | NA | NA | NA | 8.68 | 1.93 | 12.37 | 12.37 | 8.68 |
| | T | NA | NA | NA | NA | 2.05 | **1.33** | 1.53 | 1.64 | 3.85 |
| | | NA | NA | NA | NA | 29.72 | 1.93 | 14.96 | 14.96 | 8.68 |
| T44 | M | 2.05 | **1.92** | 2.05 | 3.22 | 2.93 | 2.59 | NA | NA | 2.77 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 29.72 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.37 | 2.03 | 2.16 | 2.26 | **2.03** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 16.26 | 16.26 | 8.68 |
| | T | NA | NA | NA | NA | 3.76 | **2.48** | 3.14 | 3.62 | 5.51 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 12.37 | 12.37 | 8.68 |
| HLP 320 | M | 1.11 | **1.04** | 1.11 | 1.35 | 1.64 | 1.17 | NA | NA | 1.19 |
| | | 16.26 | 14.96 | 16.26 | 16.26 | 8.68 | 2.19 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.62 | **1.13** | 1.63 | 1.98 | 1.29 |
| | | NA | NA | NA | NA | 1.93 | 2.19 | 11.07 | 11.07 | 8.68 |
| | T | NA | NA | NA | NA | 1.66 | 1.24 | **1.04** | 1.12 | 3.21 |
| | | NA | NA | NA | NA | 8.68 | 2.19 | 14.86 | 14.86 | 8.68 |
| HLP 400 | M | 0.95 | **0.89** | 0.95 | 1.15 | 1.40 | 1.04 | NA | NA | 1.04 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 8.68 | 2.19 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.40 | **1.01** | 1.39 | 1.68 | 1.11 |
| | | NA | NA | NA | NA | 1.93 | 2.19 | 11.07 | 11.07 | 8.68 |
| | T | NA | NA | NA | NA | 1.41 | 1.14 | **0.89** | 0.96 | 2.70 |
| | | NA | NA | NA | NA | 8.68 | 30.22 | 14.86 | 14.86 | 8.68 |
| GROUP 1, VEHICLE 1 | M | 3.26 | **3.06** | 3.26 | 5.12 | 4.53 | 4.43 | NA | NA | 3.90 |
| | | 14.96 | 14.96 | 14.96 | 14.96 | 8.68 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 3.79 | **3.08** | 3.63 | 4.15 | 3.17 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 12.37 | 12.37 | 8.68 |
| | T | NA | NA | NA | NA | 6.98 | 4.48 | **3.58** | 3.87 | 17.55 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 16.26 | 16.26 | 12.37 |
| GROUP 1, VEHICLE 2 | M | 2.22 | **2.08** | 2.22 | 3.49 | 3.11 | 2.98 | NA | NA | 2.70 |
| | | 14.96 | 14.96 | 14.96 | 16.16 | 8.68 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.71 | **2.26** | 2.68 | 3.03 | 2.28 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 4.09 | 3.00 | **2.28** | 2.47 | 8.11 |
| | | NA | NA | NA | NA | 11.17 | 8.68 | 16.16 | 16.16 | 11.37 |

Notes: Red values represent governing factors for each case (M, V and T).
* This 98% rating has resulted as $\sigma_{pu}$ has been used for all calculations as per AS 5100.5:2017 Clause 8.1.7 in lieu of a non-linear stress-strain curve for the strand. This methodology has been adopted to present generalised load rating results when appropriate test data is not available and approved by MRWA prior to the final design. The load rating using stress-strain tendon test data is 100%.

| Load Rating Factors Continued - Span 1 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| GROUP 1, VEHICLE 3 | M | 2.01 | **1.88** | 2.01 | 3.16 | 3.89 | 3.87 | NA | NA | 2.43 |
| | | 14.96 | 14.96 | 14.96 | 14.96 | 11.17 | 8.68 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.54 | 2.13 | 2.31 | 2.62 | **1.94** |
| | | NA | NA | NA | NA | 30.22 | 30.22 | 20.15 | 20.15 | 8.68 |
| | T | NA | NA | NA | NA | 4.04 | 2.60 | **2.15** | 2.30 | 7.58 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 16.26 | 16.26 | 11.37 |
| GROUP 1, VEHICLE 4 | M | 2.07 | **1.94** | 2.07 | 3.25 | 2.94 | 2.63 | NA | NA | 2.55 |
| | | 14.96 | 14.96 | 14.96 | 14.96 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.50 | **2.01** | 2.49 | 2.96 | 2.08 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 11.07 | 11.07 | 8.68 |
| | T | NA | NA | NA | NA | 4.00 | 2.62 | **2.16** | 2.32 | 6.15 |
| | | NA | NA | NA | NA | 8.68 | 8.68 | 14.86 | 14.86 | 8.68 |
| GROUP 2, VEHICLE 1 - 3.01 | M | 2.93 | **2.75** | 2.93 | 3.45 | 4.07 | 3.66 | NA | NA | 3.46 |
| | | 17.46 | 14.96 | 17.46 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 3.43 | 3.30 | 3.39 | 3.91 | **3.08** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 4.98 | 4.43 | **4.31** | 4.63 | 8.37 |
| | | NA | NA | NA | NA | 27.73 | 23.24 | 14.96 | 14.96 | 23.24 |
| GROUP 2, VEHICLE 1 - 3.70 | M | 2.94 | **2.73** | 2.94 | 3.46 | 4.09 | 3.56 | NA | NA | 3.42 |
| | | 17.46 | 14.96 | 17.46 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 3.46 | 3.12 | 3.42 | 3.93 | **3.02** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 4.93 | 4.54 | **4.49** | 5.56 | 8.46 |
| | | NA | NA | NA | NA | 26.53 | 23.24 | 26.53 | 26.53 | 23.14 |
| GROUP 2, VEHICLE 2 - 3.01 | M | 2.65 | **2.50** | 2.65 | 3.13 | 3.53 | 3.20 | NA | NA | 3.16 |
| | | 17.56 | 14.86 | 17.56 | 17.56 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.98 | 2.97 | 2.94 | 3.39 | **2.71** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 4.49 | 4.04 | **3.99** | 4.18 | 7.73 |
| | | NA | NA | NA | NA | 27.73 | 23.24 | 12.47 | 12.47 | 23.24 |
| GROUP 2, VEHICLE 2 - 3.70 | M | 2.72 | **2.56** | 2.72 | 3.21 | 3.66 | 3.28 | NA | NA | 3.29 |
| | | 17.56 | 14.96 | 17.56 | 17.56 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 3.09 | 3.02 | 3.04 | 3.49 | **2.78** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 4.64 | 4.08 | **3.83** | 4.03 | 7.43 |
| | | NA | NA | NA | NA | 27.73 | 23.24 | 12.47 | 12.47 | 23.24 |

Notes: Red values represent governing factors for each case (M, V and T).

| Load Rating Factors Continued - Span 1 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| GROUP 2, VEHICLE 4 | M | 2.01 | **1.91** | 2.01 | 2.37 | 2.81 | 2.46 | NA | NA | 2.42 |
| | | 17.46 | 16.26 | 17.46 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.53 | 2.27 | 2.50 | 2.87 | **2.23** |
| | | NA | NA | NA | NA | 23.24 | 1.93 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 3.42 | **2.88** | 2.98 | 3.23 | 4.07 |
| | | NA | NA | NA | NA | 27.83 | 30.22 | 16.26 | 16.26 | 30.22 |
| GROUP 2, VEHICLE 4 - NS | M | 1.70 | **1.61** | 1.70 | 2.00 | 2.37 | 2.08 | NA | NA | 2.05 |
| | | 17.46 | 16.26 | 17.46 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 2.14 | 1.92 | 2.12 | 2.43 | **1.89** |
| | | NA | NA | NA | NA | 23.24 | 1.93 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 2.89 | **2.44** | 2.53 | 2.73 | 3.44 |
| | | NA | NA | NA | NA | 27.83 | 30.22 | 16.26 | 16.26 | 30.22 |
| GROUP 2, VEHICLE 5 | M | 1.51 | **1.43** | 1.51 | 1.78 | 2.17 | 1.96 | NA | NA | 1.83 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.96 | 1.87 | 1.93 | 2.22 | **1.74** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 2.59 | **2.28** | 2.45 | 2.66 | 5.59 |
| | | NA | NA | NA | NA | 27.83 | 30.22 | 14.96 | 14.96 | 21.35 |
| GROUP 2, VEHICLE 5 - NS | M | 1.28 | **1.21** | 1.28 | 1.51 | 1.84 | 1.66 | NA | NA | 1.55 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 30.22 | 30.22 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.66 | 1.58 | 1.63 | 1.88 | **1.47** |
| | | NA | NA | NA | NA | 23.24 | 8.68 | 20.05 | 20.05 | 8.68 |
| | T | NA | NA | NA | NA | 2.19 | **1.93** | 2.07 | 2.25 | 4.73 |
| | | NA | NA | NA | NA | 27.83 | 30.22 | 14.96 | 14.96 | 21.35 |
| GROUP 2, VEHICLE 7 | M | 0.94 | **0.88** | 0.94 | 1.11 | 1.36 | 1.15 | NA | NA | 1.09 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 8.68 | 2.19 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.33 | **1.05** | 1.34 | 1.58 | 1.13 |
| | | NA | NA | NA | NA | 23.24 | 1.93 | 21.35 | 21.35 | 8.68 |
| | T | NA | NA | NA | NA | 1.49 | **1.24** | 1.28 | 1.37 | 1.73 |
| | | NA | NA | NA | NA | 30.22 | 30.22 | 16.16 | 16.16 | 30.22 |
| GROUP 2, VEHICLE 8 | M | 0.75 | **0.70** | 0.75 | 0.88 | 1.09 | 0.94 | NA | NA | 0.88 |
| | | 16.26 | 16.16 | 16.26 | 16.26 | 8.68 | 2.19 | NA | NA | 8.68 |
| | V | NA | NA | NA | NA | 1.06 | **0.85** | 1.06 | 1.24 | 0.90 |
| | | NA | NA | NA | NA | 23.24 | 1.93 | 21.35 | 21.35 | 8.68 |
| | T | NA | NA | NA | NA | 1.23 | 1.02 | **0.95** | 1.00 | 1.42 |
| | | NA | NA | NA | NA | 30.22 | 30.22 | 15.36 | 15.36 | 30.22 |

Notes: Red values represent governing factors for each case (M, V and T).

Table C.3 Bridge 1694 Span 2 Load Rating Factors (Longitudinal)

| Load Rating Factors - Span 2 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| SM1600 | M | **1.03** | **1.03** | **1.03** | 1.44 | 1.47 | 1.51 | NA | NA | 1.47 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.26 | 1.41 | **1.15** | 1.37 | 1.23 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 12.80 | 12.80 | 13.30 |
| | T | NA | NA | NA | NA | 3.20 | 4.04 | **2.37** | 2.68 | 6.82 |
| | | NA | NA | NA | NA | 14.10 | 1.93 | 14.10 | 14.10 | 14.10 |
| T44 | M | **1.57** | **1.57** | **1.57** | 2.44 | 2.39 | 2.56 | NA | NA | 2.19 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 3.50 |
| | V | NA | NA | NA | NA | 1.77 | 2.41 | **1.62** | 1.95 | 1.75 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 12.80 | 12.80 | 13.30 |
| | T | NA | NA | NA | NA | 3.75 | 4.93 | **2.95** | 3.38 | 6.91 |
| | | NA | NA | NA | NA | 15.30 | 1.93 | 14.10 | 14.10 | 15.30 |
| HLP 320 | M | **1.16** | **1.16** | **1.16** | 1.36 | 1.83 | 1.99 | NA | NA | 1.88 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | **1.05** | 3.25 | 1.57 | 1.96 | 1.20 |
| | | NA | NA | NA | NA | 1.93 | 3.80 | 13.30 | 13.30 | 1.93 |
| | T | NA | NA | NA | NA | 2.69 | **2.25** | 2.45 | 3.18 | 5.82 |
| | | NA | NA | NA | NA | 13.30 | 18.89 | 12.80 | 12.80 | 13.30 |
| HLP 400 | M | **1.01** | **1.01** | **1.01** | 1.19 | 1.59 | 1.72 | NA | NA | 1.63 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.55 | 2.80 | **1.36** | 1.69 | 1.58 |
| | | NA | NA | NA | NA | 1.93 | 3.70 | 13.30 | 13.30 | 1.93 |
| | T | NA | NA | NA | NA | 1.91 | 1.75 | **1.75** | 2.26 | 4.10 |
| | | NA | NA | NA | NA | 13.30 | 18.89 | 13.30 | 13.30 | 13.30 |
| GROUP 1, VEHICLE 1 | M | **2.85** | **2.85** | **2.85** | 4.43 | 4.11 | 4.49 | NA | NA | 3.83 |
| | | 9.10 | 9.10 | 9.10 | 9.10 | 1.93 | 1.93 | NA | NA | 3.90 |
| | V | NA | NA | NA | NA | 3.29 | 3.98 | **3.04** | 3.66 | 3.21 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 12.80 | 12.80 | 13.30 |
| | T | NA | NA | NA | NA | 5.70 | 11.77 | **4.29** | 4.54 | 11.88 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 12.80 | 12.80 | 15.30 |
| GROUP 1, VEHICLE 2 | M | **2.13** | **2.13** | **2.13** | 3.31 | 2.93 | 3.12 | NA | NA | 2.95 |
| | | 9.00 | 9.10 | 9.00 | 9.00 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.79 | 2.84 | **2.40** | 2.97 | 2.87 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 13.30 | 13.30 | 1.93 |
| | T | NA | NA | NA | NA | 4.71 | 7.60 | **2.73** | 2.80 | 5.19 |
| | | NA | NA | NA | NA | 18.89 | 1.93 | 6.40 | 6.40 | 18.89 |

Notes: Red values represent governing factors for each case (M, V and T).

| Load Rating Factors Continued - Span 2 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| GROUP 1, VEHICLE 3 | M | **1.91** | **1.91** | **1.91** | 2.96 | 2.50 | 2.68 | NA | NA | 2.52 |
| | | 9.00 | 9.10 | 9.00 | 9.00 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.16 | 2.60 | **2.00** | 2.48 | 2.12 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 3.97 | 4.63 | **2.24** | 2.32 | 4.87 |
| | | NA | NA | NA | NA | 7.60 | 1.93 | 7.70 | 7.70 | 18.89 |
| GROUP 1, VEHICLE 4 | M | **2.04** | **2.04** | **2.04** | 3.18 | 2.74 | 2.96 | NA | NA | 2.77 |
| | | 9.00 | 9.10 | 9.00 | 9.00 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.39 | 2.75 | **2.20** | 2.65 | 2.33 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 12.80 | 12.80 | 13.30 |
| | T | NA | NA | NA | NA | 4.30 | 7.68 | 3.17 | **3.17** | 10.38 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 9.00 | 9.00 | 13.30 |
| GROUP 2, VEHICLE 1 - 3.01 | M | **2.72** | **2.72** | **2.72** | 3.17 | 3.74 | 3.65 | NA | NA | 3.62 |
| | | 9.00 | 9.00 | 9.00 | 9.00 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 3.24 | 3.38 | 3.04 | 3.88 | **2.80** |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 4.76 | 4.20 | **3.63** | 3.91 | 9.81 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 9.40 | 9.40 | 13.30 |
| GROUP 2, VEHICLE 1 - 3.70 | M | **2.72** | **2.72** | **2.72** | 3.17 | 3.77 | 3.77 | NA | NA | 3.70 |
| | | 9.00 | 9.00 | 9.00 | 9.00 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 3.27 | 3.54 | 3.08 | 3.86 | **3.08** |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 12.80 |
| | T | NA | NA | NA | NA | 4.66 | 4.74 | **3.73** | 4.12 | 10.38 |
| | | NA | NA | NA | NA | 12.80 | 1.93 | 11.50 | 11.50 | 13.30 |
| GROUP 2, VEHICLE 2 - 3.01 | M | **2.40** | **2.40** | **2.40** | 2.80 | 3.25 | 3.15 | NA | NA | 3.14 |
| | | 9.10 | 9.10 | 9.10 | 9.10 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.74 | 3.00 | 2.58 | 3.28 | **2.39** |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 3.46 | 4.03 | **3.01** | 3.36 | 7.44 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 11.50 | 11.50 | 13.30 |
| GROUP 2, VEHICLE 2 - 3.70 | M | **2.50** | **2.50** | **2.50** | 2.91 | 3.36 | 3.29 | NA | NA | 3.29 |
| | | 9.10 | 9.10 | 9.10 | 9.10 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.86 | 3.20 | 2.68 | 3.39 | **2.51** |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 3.64 | 4.73 | **3.22** | 3.62 | 7.83 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 11.50 | 11.50 | 13.30 |

Notes: Red values represent governing factors for each case (M, V and T).

| Load Rating Factors Continued - Span 2 | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Vehicle | Factor | $k_{M1}$ | $k_{M2}$ | $k_{M3}$ | $k_{M4}$ | $k_{V1}$ | $k_{V2}$ | $k_{V3}$ | $k_{V4}$ | $k_{V5}$ |
| | Distance from Abut. 1 CL (m) | $d_{M1}$ | $d_{M2}$ | $d_{M3}$ | $d_{M4}$ | $d_{V1}$ | $d_{V2}$ | $d_{V3}$ | $d_{V4}$ | $d_{V5}$ |
| GROUP 2, VEHICLE 4 | M | **1.74** | **1.74** | **1.74** | 2.03 | 2.68 | 2.67 | NA | NA | 2.66 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.34 | 2.49 | **2.19** | 2.73 | 2.31 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 12.30 |
| | T | NA | NA | NA | NA | 4.38 | 3.89 | 3.10 | **3.10** | 10.90 |
| | | NA | NA | NA | NA | 18.89 | 1.93 | 10.20 | 10.20 | 13.30 |
| GROUP 2, VEHICLE 4 - NS | M | **1.47** | **1.47** | **1.47** | 1.72 | 2.27 | 2.26 | NA | NA | 2.25 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.98 | 2.11 | **1.85** | 2.31 | 1.96 |
| | | NA | NA | NA | NA | 13.30 | 1.93 | 13.30 | 13.30 | 12.30 |
| | T | NA | NA | NA | NA | 3.71 | 3.29 | **2.62** | 2.62 | 9.22 |
| | | NA | NA | NA | NA | 18.89 | 1.93 | 10.20 | 10.20 | 13.30 |
| GROUP 2, VEHICLE 5 | M | **1.33** | **1.33** | **1.33** | 1.55 | 2.02 | 1.96 | NA | NA | 1.98 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 2.02 | 1.93 | 1.79 | 2.28 | **1.64** |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 2.84 | 2.47 | 1.91 | **1.91** | 7.63 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 11.40 | 11.40 | 13.30 |
| GROUP 2, VEHICLE 5 - NS | M | **1.12** | **1.12** | **1.12** | 1.31 | 1.71 | 1.66 | NA | NA | 1.68 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.71 | 1.63 | 1.51 | 1.93 | **1.39** |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 13.30 | 13.30 | 13.30 |
| | T | NA | NA | NA | NA | 2.40 | 2.09 | 1.62 | **1.62** | 6.46 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 11.40 | 11.40 | 13.30 |
| GROUP 2, VEHICLE 7 | M | **0.93** | **0.93** | **0.93** | 1.09 | 1.43 | 1.50 | NA | NA | 1.45 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.44 | 1.46 | **1.28** | 1.59 | 1.47 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 13.30 | 13.30 | 1.93 |
| | T | NA | NA | NA | NA | 2.32 | 2.17 | **1.61** | 1.61 | 6.39 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 10.40 | 10.40 | 13.30 |
| GROUP 2, VEHICLE 8 | M | **0.76** | **0.76** | **0.76** | 0.88 | 1.16 | 1.22 | NA | NA | 1.18 |
| | | 10.30 | 10.30 | 10.30 | 10.30 | 1.93 | 1.93 | NA | NA | 1.93 |
| | V | NA | NA | NA | NA | 1.17 | 1.19 | **1.04** | 1.29 | 1.20 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 13.30 | 13.30 | 1.93 |
| | T | NA | NA | NA | NA | 1.62 | 1.53 | **1.05** | 1.05 | 4.32 |
| | | NA | NA | NA | NA | 1.93 | 1.93 | 10.20 | 10.20 | 13.30 |

Notes: Red values represent governing factors for each case (M, V and T).

### C.2.9 Other Superstructure Checks

#### Interface Shear

Interface shear between deck slab and tee-roff beams was checked for both longitudinal and transverse design shear actions in accordance with Clause 8.4 in AS 5100.5:2017 and was found not to govern the load ratings. The anchored N12-150 U bars in the flanges were used in the transverse interface shear assessment. The N12-150 U bars were used together with the shear/torsion reinforcement in the longitudinal interface shear assessment. Group 2 Vehicle 8 governed both longitudinal and transverse design actions. The interface shear results are shown in the following Table C.4.

Table C.4 Interface Shear for Group 2 Vehicle 8 (as per AS 5100.5:2017)

| **Interface Shear - Longitudinal** | **Symbol** | **PE+CE** | **LL** | **Units** |
|---|---|---|---|---|
| ULS Design Shear | V* | 1346 | 2884 | kN |
| Shear Flow | $q^*$ | 544 | 1165 | kN/m |
| Capacity | $q_{ALLOW}$ | 4541 | | kN/m |
| Load Rating | %LR | **343%** | | % |
| | | | | |
| **Interface Shear - Transverse** | **Symbol** | **SDL+CE** | **LL** | **Units** |
| ULS Design Shear | V* | 6 | 184 | kN |
| Shear Flow | $q^*$ | 27 | 818 | kN/m |
| Capacity | $q_{ALLOW}$ | 14565 | | kN/m |
| Load Rating | %LR | **1778%** | | % |

#### Transverse Bending & Shear

Bridge No. 1694 has sufficiently anchored interface shear reinforcement between the deck slab and tee-roff flange and therefore a 300 mm thick composite section was used in transverse hog bending and shear – refer Figure C.9 below. Critical hog bending was located where the flange meets the web of the beam.

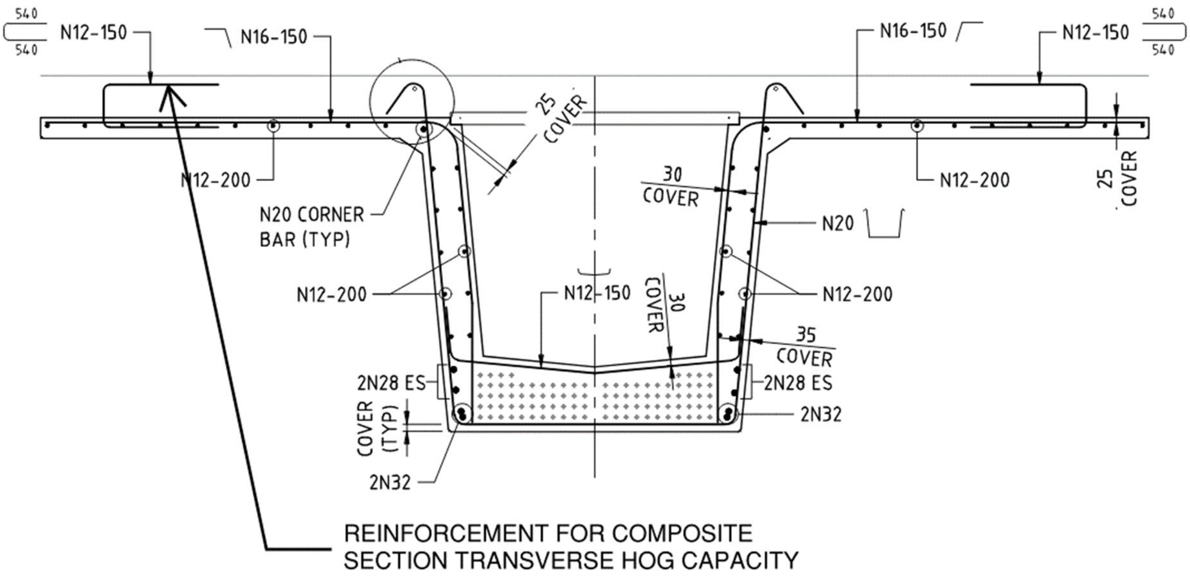



Figure C.9 Transverse Hog Reinforcement

For transverse sag bending check at mid-span between tee-roffs, a 200 mm thick deck slab was used for capacity considering the discontinuity of adjacent flanges. This was the critical location for transverse checks, governed by Group 2 Vehicle 8. The results are shown in Table C.5.

Table C.5 Critical Transverse Check for Group 2 Vehicle 8 (as per AS 5100.5:2017)

| **Transverse Bending** | **Symbol** | **SDL** | **LL** | **Units** |
|---|---|---|---|---|
| ULS Bending Moment | $M^*$ | -6.6 | 177.6 | kNm |
| Corresponding Shear | $V_{CORR}$ | -0.3 | 40.2 | kN |
| Total Force in Longitudinal Reinforcement | $F_{td}$ | 1627 | | kN |
| Total Available Force | $F_{AVAIL}$ | 1640 | | kN |
| Transverse Load Rating | %LR | **101%** | | % |
| Longitudinal Load Rating (for Comparison) | %LR | 72% | | % |

Note: PE effects were in the opposite direction at the critical location and therefore ignored.

Transverse bending and shear were checked in line with the MCFT approach considering shear and bending interaction, complying with AS 5100.5:2017. Transverse design actions were found not to govern the load ratings.

### Compression Fan Regions

Locations within $d_o$ from face of supports are reinforced with adequate shear reinforcement to transfer stresses via compression fan-shaped strut-and-tie action. These compression fan regions were checked using strut and tie modelling (shown in Figure C.10 below) and did not govern the load ratings.

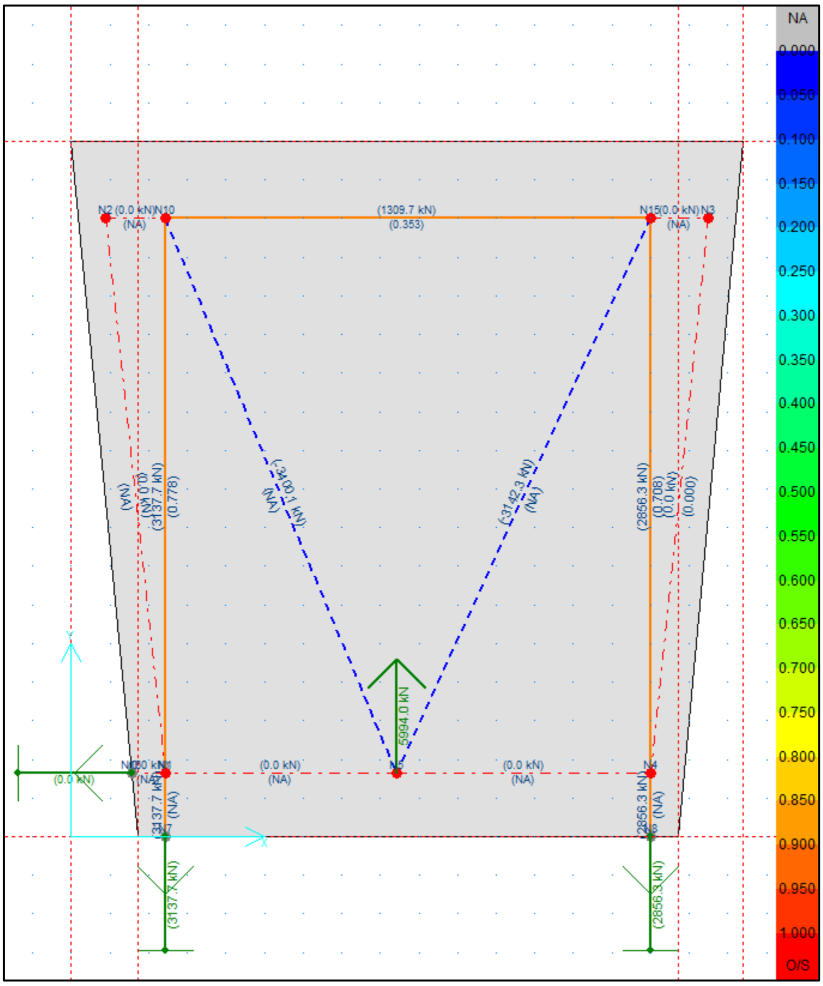



Figure C.10 End Diaphragm Strut & Tie Model

Bearing No. 3 was critical for maximum PE + SM1600 loading. The load rating is shown in Table C.6 below.

Table C.6 End Diaphragm Check (as per AS 5100.5:2017)

| Pier Pilecap | Symbol | PE+LL | PE | LL | Units |
|---|---|---|---|---|---|
| ULS Bearing Load | $N^*$ | 5994 | 1838 | 4156 | kN |
| Vertical Reinforcement Utilisation | | 0.778 | 0.238 | 0.540 | |
| Load Rating | %LR | | **141%** | | % |

### C.2.10 Detailed Substructure Checks

***Note: Detailed substructure check reporting is optional when substructure rating is significantly higher than the superstructure load ratings. Example given here is only for demonstration purpose.***

The bearings, capbeam, columns, footings and piles were checked and found not to govern the load ratings. Please note this section is only required when substructure rating governs for design vehicles, or the substructure rating is requested.

### Bearings

Loads on the bearings with corresponding rotations were checked against AS 5100.4:2017. Maximum compressive stress in accordance with clause 12.6.2 controlled the bearings on span 1. The SM1600 loading results on bearing part No. 161212C are shown in Table C.7 below.

Table C.7 Load Rating Check of Elastomeric Bearings (as per AS 5100.4:2017)

| **Compressive Stress** | **Symbol** | **PE** | **LL** | **Units** |
|---|---|---|---|---|
| SLS Vertical Bearing Load | N | 1470 | 2309 | kN |
| Bonded Area of Bearing | $A_b$ | 255176 | 255176 | $mm^2$ |
| Mean Compressive Stress | $\sigma_{N/Ab}$ | 5.761 | 9.049 | MPa |
| Compressive Stress Limit | $\sigma_{max}$ | 15.00 | | MPa |
| Load Rating | %LR | **102%** | | % |
| | | | | |
| **Shear Strain** | **Symbol** | **PE** | **LL** | **Units** |
| Rotation - Longitudinal | $\alpha_a$ | 0.00272 | 0.00428 | Radians |
| Rotation - Transverse | $\alpha_b$ | 0.00194 | 0.00306 | Radians |
| Vector Sum | $\alpha$ | 0.00335 | 0.00526 | Radians |
| Compressive Strain | $\varepsilon_c$ | 0.011 | 0.017 | |
| Shear Strain – Vertical | $e_{sc}$ | 0.761 | 1.196 | |
| Shear Strain – Rotation | $e_{sr}$ | 0.290 | 0.456 | |
| Shear Strain – Tangential | $e_{sh}$ | 0.000 | 0.239 | |
| Shear Strain – Total | $\varepsilon$ | 1.052 | 1.891 | |
| Shear Strain – Limit | $\varepsilon_{max}$ | 3.130 | | |
| Load Rating | %LR | **110%** | | % |

### Abutment Capbeam & Columns

A global model was used to analyse the Abutments, shown in Figure C.11 below. Bursting stresses at bearing locations were also checked in accordance with the BBR Multi-wire System prestressing (and bearings) manual (BBR Australia Pty Ltd, 1973).

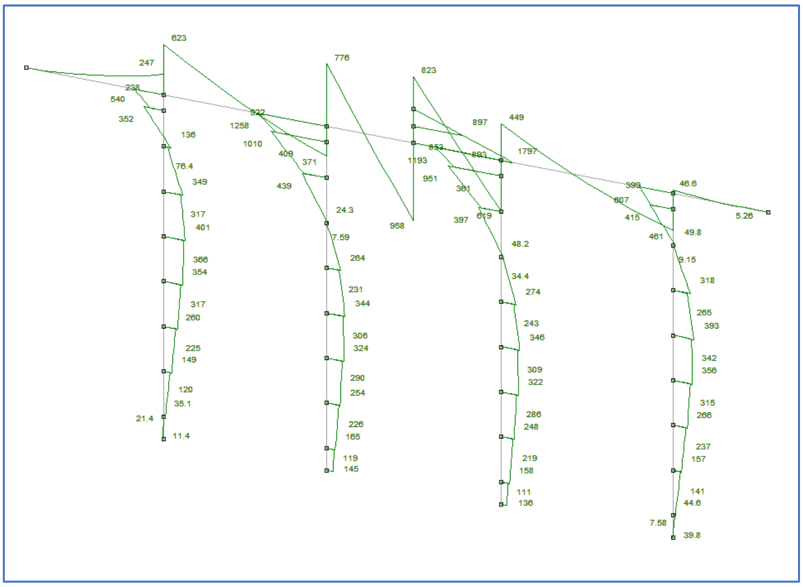


Figure C.11 Abutment 1 Capbeam & Columns with Lateral MSE Stiffnesses – EQ Transverse Moment Diagram (Note Shear Key Full Extent Between Columns 2 & 3)

ULS maximum PE and transverse EQ governed both the capbeam and column bending load ratings, while maximum PE and SM1600 governed the capbeam bursting and spalling ratings, which are summarised in Table C.8 below.

Table C.8 Load Rating of Abutment Capbeam and Columns (as per AS 5100.5:2017 and BBR Multi-wire System: Prestressing Manual 1973)

| **Capbeam Bending At Pile 1** | **Symbol** | **PE** | **EQ** | **Units** |
|---|---|---|---|---|
| ULS Moment | $-M^*$ | -49.5 | -573.5 | kNm |
| Corresponding Shear | $V_{Corr}$ | 46.2 | 188.8 | kN |
| Corresponding Torsion | $T_{Corr}$ | 11.9 | 191.1 | kNm |
| Load Rating | %LR | **113%** | | % |
| Governing Check | | M2 | | |
| | | | | |
| **Capbeam Bending At Pile 2** (-$M^*$ Check Adjacent Shear Key) | **Symbol** | **PE** | **EQ** | **Units** |
| ULS Moment | $-M^*$ | -40.8 | -546.6 | kNm |
| ULS Bending Capacity | $\phi M_u$ | -889.0 | | kNm |
| Load Rating | %LR | **155%** | | % |
| Governing Check | | M1 | | |
| | | | | |
| **Capbeam Bursting** | **Symbol** | **PE** | **LL** | **Units** |
| ULS Bearing Load | $N^*$ | 1838 | 4156 | kN |
| Tensile Capacity of Bars Provided (Each Direction) | $\phi N$ | 880 | | kN |
| Bursting Forces | $T^*_b$ | 259 | 586 | kN |
| Load Rating | %LR | **106%** | | % |
| | | | | |
| **Capbeam Spalling** | **Symbol** | **PE** | **LL** | **Units** |
| ULS Bearing Loads | $N^*$ | 1838 | 4156 | kN |
| Tensile Capacity of Top Longitudinal Bars Provided | $\phi N$ | 1488 | | kN |
| Spalling Forces | $T^*_s$ | 418 | 944 | kN |
| Load Rating | %LR | **113%** | | % |
| | | | | |
| **Columns – Biaxial Bending** | **Symbol** | **PE** | **EQ** | **Units** |
| -M*x | $-M^*_x$ | -14.0 | -266.0 | kNm |
| -M*y | $-M^*_y$ | -83.9 | -926.1 | kNm |
| ϕMux | $\phi M_{ux}$ | 1604 | | kNm |
| ϕMuy | $\phi M_{uy}$ | 1237 | | kNm |
| N* | $N_{Corr}$ | -1890 | -371 | kN |
| ϕNuo | $\phi N_{uo}$ | 12870 | | kN |
| αn | $\alpha_n$ | 1.00 | | |
| Utilisation | | 0.077 | 0.915 | |
| Load Rating | %LR | **101%** | | % |

**Pad Footings**

Bearing reactions from the superstructure grillage (shown in Figure C.12 below) and hand calculations were used to determine abutment pad footing bearing pressures. Bearing capacities are from Geotechnical Report 10-0107-030-BR-RP-0001-00.

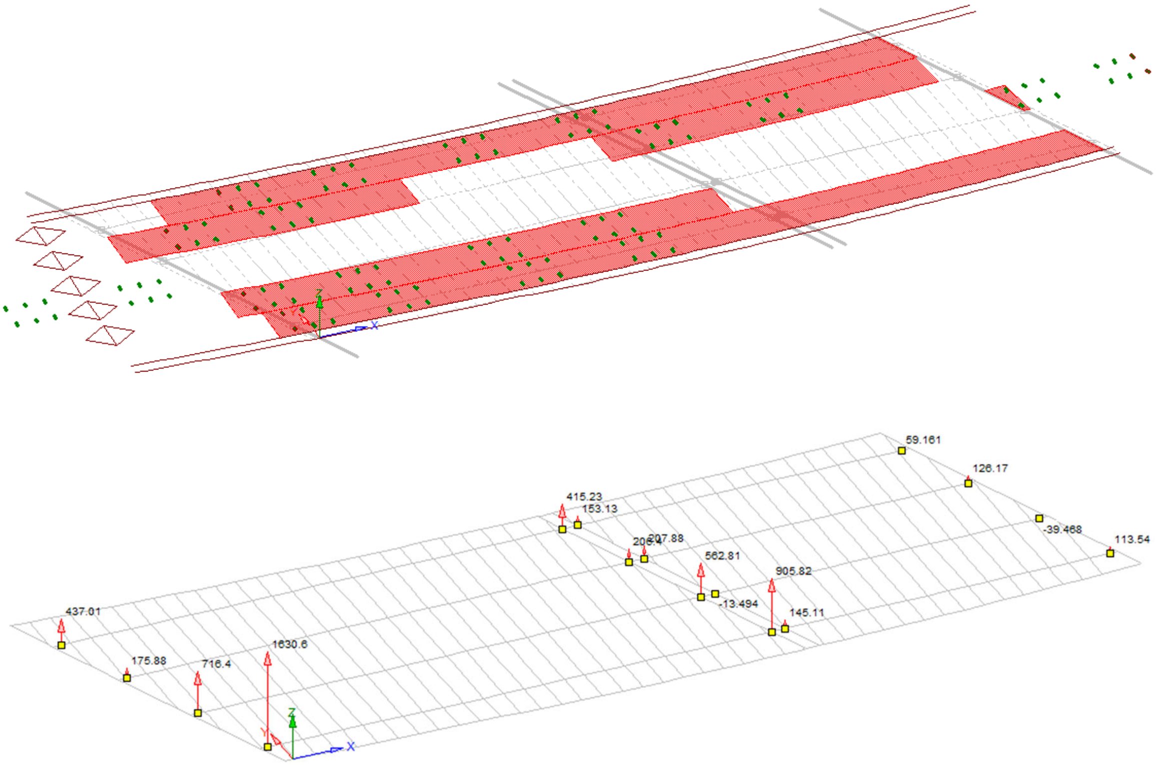



Figure C.12 SBD Model – SLS SM1600 Bearing Reactions for Pad Footing Bearing Check

SLS Abutment 1 footing pressures for SM1600 with transverse wind design actions were checked against acceptable settlements provided in Geotechnical Report 10-0107-030-BR-RP-0001-00. ULS pressures were also checked against ultimate bearing capacity for the SM1600 transverse bending case, however, maximum PE and longitudinal EQ case governed the ULS rating. ULS bearing capacities were derived with the Meyerhof method using soil parameters from the geotechnical report. The load ratings are shown in Table C.9 below.

Table C.9 Critical Pad Footing Bearing Check for Abutment 1 (as per AS 5100.5:2017)

| **Abutment 1 Pad Footing Bearing - SLS** | **Symbol** | **PE** | **LL** | **0.7*Wind** | **Units** |
|---|---|---|---|---|---|
| Bearing 1 Reaction | $N_1$ | - | 437 | - | kN |
| Bearing 2 Reaction | $N_2$ | - | 176 | - | kN |
| Bearing 3 Reaction | $N_3$ | - | 716 | - | kN |
| Bearing 4 Reaction | $N_4$ | - | 1631 | - | kN |
| Total Vertical Force | $N_{SLS}$ | 23733 | 2960 | 0 | kN |
| Transverse Force | $F_{T,SLS}$ | 0 | 0 | 18 | kN |
| Longitudinal Force | $F_{L,SLS}$ | 0 | 180 | 0 | kN |
| Transverse Moment | $M_{T.SLS}$ | 233 | 11847 | 153 | kNm |
| Longitudinal Moment | $M_{L,SLS}$ | | 7080 | | kNm |
| H/V Ratio | | | 0.007 | | |
| Transverse Eccentricity | $e_T$ | | 0.458 | | m |
| Longitudinal Eccentricity | $e_L$ | | 0.265 | | m |
| Effective Length | L' | | 26.383 | | m |
| Effective Width | B' | | 3.770 | | m |
| SLS Bearing Pressure | $q_{SLS}$ | 239 | 30 | 0 | kPa |
| H/V Reduction Factor | | | 0.983 | | |
| SLS Required Bearing Capacity | $q_{SLS,REQ'D}$ | | 273 | | kPa |
| Load Rating | %LR | | **100%** | | % |

| **Abutment 1 Pad Footing Bearing – ULS Transverse SM1600 + Wind** | **Symbol** | **PE+LL** | **PE** | **LL** | **Units** |
|---|---|---|---|---|---|
| Total Vertical Force | $N^*$ | 34796 | 29468 | | kN |
| Transverse Force | $F_{T,ULS}$ | 22 | 0 | | kN |
| Longitudinal Force | $F_{L,ULS}$ | 324 | 0 | | kN |
| Transverse Moment | $M_{T.ULS}$ | 21764 | 276 | | kNm |
| Longitudinal Moment | $M_{L,ULS}$ | 11218 | 3199 | | kNm |
| H/V Ratio | | 0.009 | 0.000 | | |
| Transverse Eccentricity | $e_T$ | 0.625 | 0.009 | | m |
| Longitudinal Eccentricity | $e_L$ | 0.322 | 0.109 | | m |
| Effective Length | L' | 26.049 | 27.281 | | m |
| Effective Width | B' | 3.655 | 4.083 | | m |
| ULS Bearing Pressure | $q^*$ | 365 | 265 | 101 | kPa |
| ULS Bearing Capacity (Meyerhof) | $\phi q_{ULT}$ | 598 | | | kPa |
| Load Rating | %LR | | **330%** | | % |

| **Abutment 1 Pad Footing Bearing – ULS Longitudinal EQ** | **Symbol** | **PE+EQ** | **PE** | **EQ** | **Units** |
|---|---|---|---|---|---|
| Total Vertical Force | $N^*$ | 29468 | 29468 | - | kN |
| Transverse Force | $F_{T,ULS}$ | 0 | 0 | - | kN |
| Longitudinal Force | $F_{L,ULS}$ | 2248 | 0 | - | kN |
| Transverse Moment | $M_{T.ULS}$ | 276 | 276 | - | kNm |
| Longitudinal Moment | $M_{L,ULS}$ | 20022 | 3199 | - | kNm |
| H/V Ratio | | 0.076 | 0.000 | - | |
| Transverse Eccentricity | $e_T$ | 0.009 | 0.009 | - | m |
| Longitudinal Eccentricity | $e_L$ | 0.679 | 0.109 | - | m |
| Effective Length | L' | 27.281 | 27.281 | - | m |
| Effective Width | B' | 2.941 | 4.083 | - | m |
| ULS Bearing Pressure | $q^*$ | 367 | 265 | 103 (eqv.) | kPa |
| ULS Bearing Capacity (Meyerhof) | $\phi q_{ULT}$ | 468 | - | - | kPa |
| Load Rating | %LR | | **198%** | | % |

Similarly, for the structural capacity of the footing, the SM1600 transverse case governed in the transverse direction (transverse to bridge) and the maximum PE and longitudinal EQ in the longitudinal direction. For the structural check the footing was modelled using Strand7, shown in Figure C.13 below.

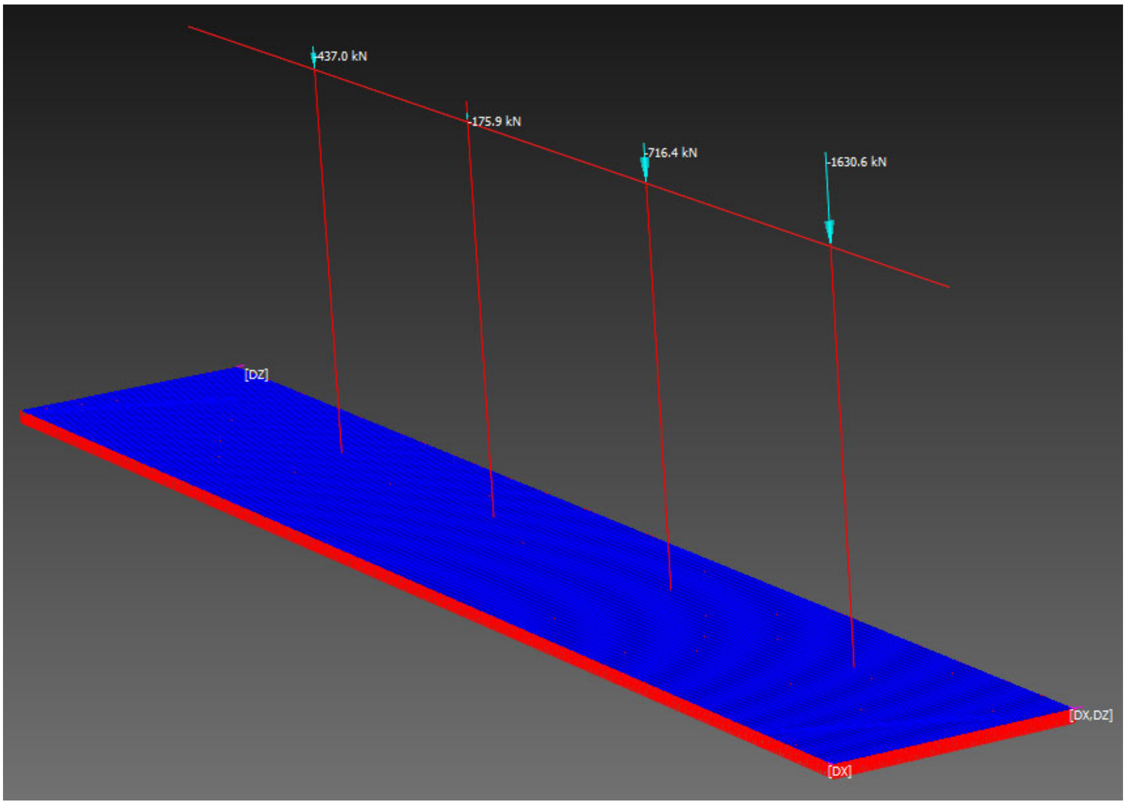



Figure C.13 Strand7 Model for Abutment 1 Pad Footing Structural Check

The structural check of Abutment 1 pad footing is shown in Table C.10 below.

Table C.10 Critical Pad Footing Structural Check for Abutment 1 (as per AS 5100.5:2017)

| **Abutment 1 Pad Footing Structural – ULS Transverse SM1600 + Wind** | **Symbol** | **PE+LL +0.7*Wind** | **PE +0.7*Wind** | **LL** | **Units** |
|---|---|---|---|---|---|
| Maximum Moment | $M^*$ | 1058 | 542 | - | kNm/m |
| Corresponding Shear | $V_{Corr}$ | 527 | 228 | - | kN/m |
| Bottom Reinforcement Utilisation - MCFT | | 0.96 | 0.38 | 0.58 (equiv.) | |
| Load Rating | %LR | **107%** | | | % |
| | | | | | |
| **Abutment 1 Pad Footing Structural – ULS Longitudinal EQ** | **Symbol** | **PE+EQ** | **PE** | **EQ** | **Units** |
| Maximum Moment | $M^*$ | 444 | 186 | - | kNm/m |
| Corresponding Shear | $V_{Corr}$ | 583 | 283 | - | kN/m |
| Bottom Reinforcement Utilisation - MCFT | | 0.98 | 0.27 | 0.71 (equiv.) | |
| Load Rating | %LR | **103%** | | | % |

**Pier Columns**

Hand calculations and a Strand7 model (shown in Figure C.14 below) were used to check worst case design actions on the pier columns. Bursting stresses at bearing locations were also checked in accordance with the BBR Multi-wire System prestressing (and bearings) manual (BBR Australia Pty Ltd, 1973).

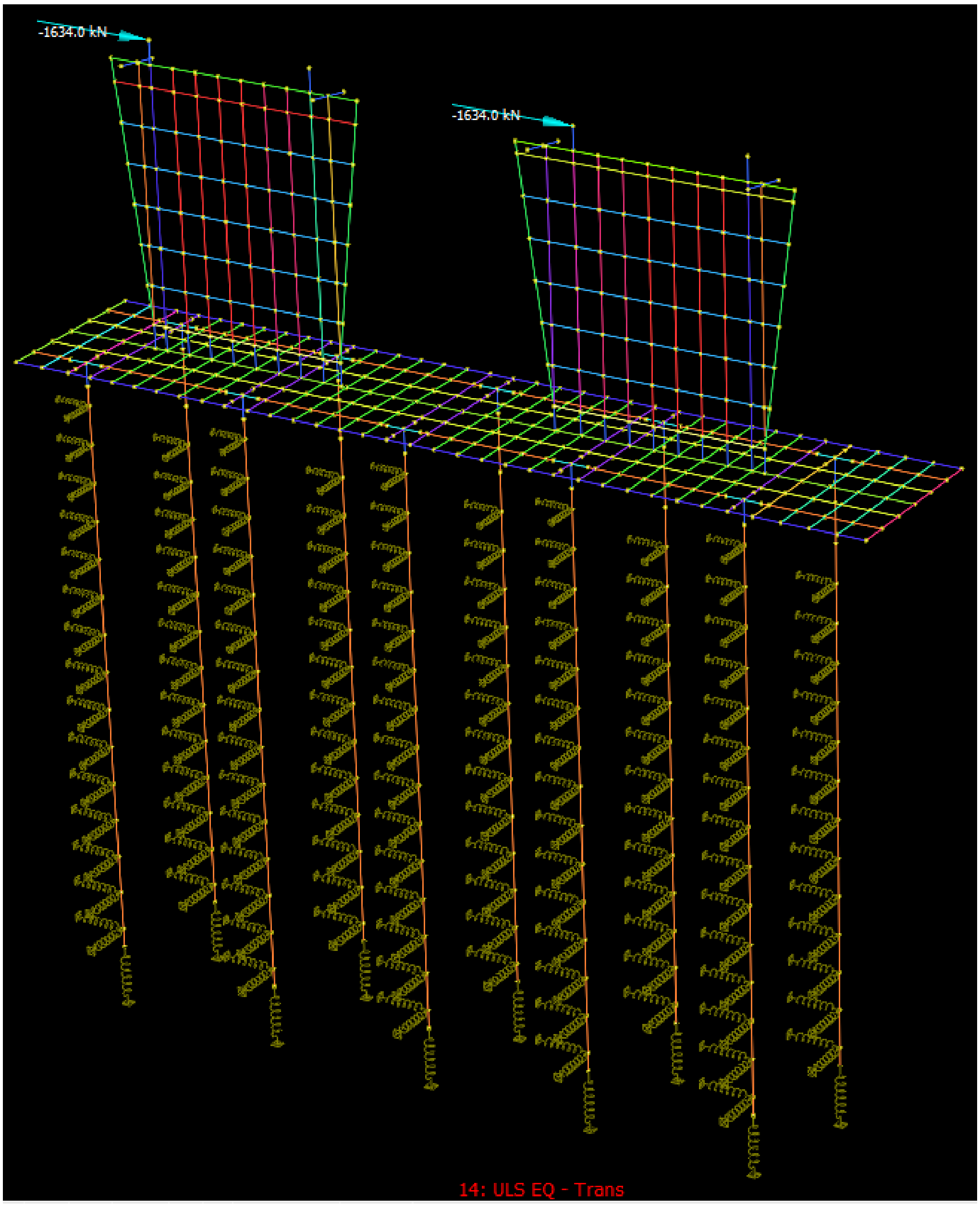



Figure C.14 Pier Strand7 Model (EQ Transverse Case)

ULS maximum PE + transverse EQ governed the pier column load ratings. The check conservatively ignored the shear keys at the abutments. The results are shown in Table C.11 below.

Table C.11 Pier Column Check (as per AS 5100.5:2017 and BBR Multi-wire System: Prestressing Manual 1973)

| **Pier Columns** | **Symbol** | **PE** | **EQ** | **Units** |
|---|---|---|---|---|
| EQ Mass at Pier | $m_P$ | - | 1063.1 | t |
| Horizontal Coefficient | $C_d(Tf)_P$ | - | 0.2153 | |
| EQ Mass to Abutment 1 | $m_{A1}$ | - | 600.0 | t |
| Horizontal Coefficient $C_d(Tf)_{A1}$ | $C_d(Tf)_{A1}$ | - | 0.1076 | |
| EQ Mass to Abutment 2 | $m_{A2}$ | - | 415.3 | t |
| Horizontal Coefficient $C_d(Tf)_{A2}$ | $C_d(Tf)_{A2}$ | - | 0.1076 | |
| Stiffness Proportion to Pier Column | % | - | 49.3% | % |
| Earthquake Force to Pier Column | $F_{EQ}$ | - | 1634 | kN |
| Height of Mass | $h_P$ | - | 7.11 | m |
| Transverse Bending Moment | $M_{x,EQ}$ | 355 | 11618 | kNm |
| Transverse Pier Bending Capacity | $\phi M_{ux}$ | 135315 | | kNm |
| Load Rating | %LR | **1162%** | | kNm |
| | | | | |
| **Pier Bursting** | **Symbol** | **PE** | **LL** | **Units** |
| ULS Bearing Load | $N^*$ | 1796 | 2684 | kN |
| Bursting Force Resisted + Capacity of Ties Provided (Each Direction) | $\phi T_b$ | 1049 | | kN |
| Bursting Forces | $T^*_b$ | 244 | 365 | kN |
| Load Rating | %LR | **221%** | | |
| | | | | |
| **Pier Spalling** | **Symbol** | **PE** | **LL** | **Units** |
| ULS Bearing Loads (Average of Span 1 & Span 2) | $N^*$ | 2138 | 1389 | kN |
| Spalling Force Resisted + Capacity of Top Longitudinal Bars Provided | $\phi T_b$ | 2015 | | |
| Spalling Forces | $T^*_b$ | 325 | 211 | kN |
| Load Rating | %LR | **799%** | | % |

## Pier Pilecap

A strut and tie model was used to assess the pilecap via transfer of loads from the pier columns to the piles. The model is shown in Figure C.15 below.

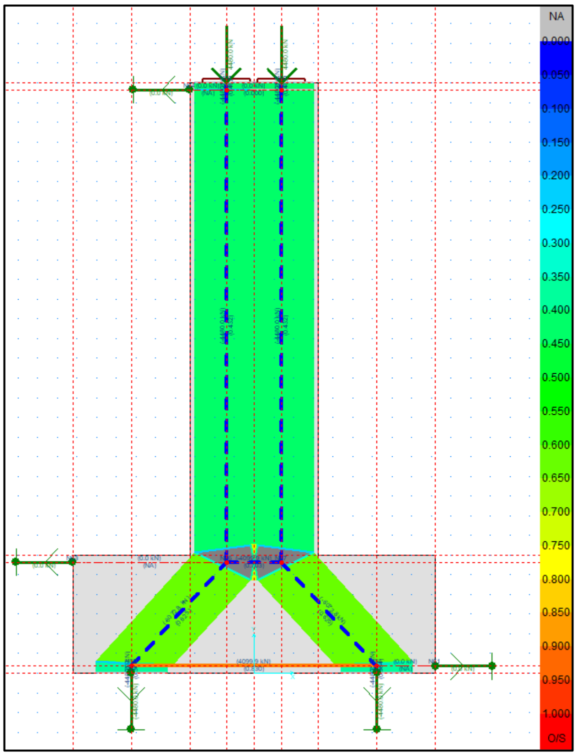


Figure C.15 CAST Strut & Tie Model – Pier Pilecap

Maximum PE and SM1600 loading governed the load rating of the bottom reinforcement. The load rating is shown in Table C.12 below.

Table C.12 Pier Pilecap Check (as per AS 5100.5:2017)

| **Pier Pilecap** | **Symbol** | **PE+LL** | **PE** | **LL** | **Units** |
|---|---|---|---|---|---|
| ULS Bearing Load | $N^*$ | 4480 | 1796 | 2684 | kN |
| Bottom Reinforcement Utilisation | | 0.830 | 0.333 | 0.497 | |
| Load Rating | %LR | | **134%** | | % |

### Pier Piles

Design actions were obtained from the Strand7 model shown in Figure C.14 above, with minimum PE and longitudinal impact (longitudinal to bridge) to the leading edge of Pier 1 governing. The load rating is shown in Table C.13 below.

Table C.13 Pier Piles Check (as per AS 5100.5:2017)

| **Pier Pile** | **Symbol** | **PE+LL** | **PE** | **LL** | **Units** |
|---|---|---|---|---|---|
| ULS Maximum Moment | $M^*$ | 1124 | 50 | - | kNm |
| Corresponding Shear | $V_{Corr}$ | 196 | 31 | - | kN |
| Corresponding Axial Force | $N_{Corr}$ | -821 | -1464 | - | kN |
| Long. Reinforcement Utilisation | | 0.99 | 0.01 | 0.98 | |
| Load Rating | %LR | | **101%** | | % |

### C.2.11 Conclusion and Recommendations

SM1600, T44, HLP, Group 1 and Group 2 vehicles were load rated for bending and shear using the MCFT approach in AS 5100.5:2017. The grillage properties were determined in line with Bridge Deck Behaviour (Hambly, 1991). The assessment covered 4No. bending checks and 5 No. shear checks for longitudinal design actions at 0.1 m increments, for 3 no. cases (a) maximum bending, corresponding shear and corresponding torsion; (b) maximum shear, corresponding bending and corresponding torsion; and (c) maximum torsion, corresponding bending and corresponding shear. The increments accounted for changes in section properties along the beams, as well as a worst-case combination of bending, shear and torsion. The design actions were assessed iteratively against bending and shear capacities. Compression fan regions, transverse bending, interface shear, substructure, foundations and bearings were also checked and found not to govern the load ratings.

Bridge No. 1694 is only 2 years old and there are no recommendations at this stage.

# SECTION 5 – DESIGN VEHICLE LOADINGS

This information is Section 5 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Engineer Bridge Loading is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

## Authorisation

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 04/09/2023

**Document No: 3912/02-5**

**Controlled Copies shall be marked accordingly**

## SECTION 5

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| 6 | 1 | 20/02/04 | Road Train Vehicles diagram added to Appendix A |
| All | 2 | 19/12/05 | Complete review for introduction of AS 5100 |
| 3 | 3 | 10/01/07 | Amendment of Document Number of High Wide Loads |
| 3 | 4 | 11/01/12 | Amendment of first paragraph in Section 5.2 |
| All | 5 | 06/07/22 | Complete review for introduction of AS5100-2017 |
| 3 | 6 | 04/09/23 | Amendment of Design Vehicle Loading Width and Service Vehicle |
| | | | |
| | | | |
| | | | |
| | | | |
| | | | |

Custodian Endorsement

R Hossain
Engineer Bridge Loading
Date: 04/09/2023

# 5 DESIGN VEHICLE LOADINGS

## 5.1 Introduction

Generally vehicular (and pedestrian) loadings to be applied for the design of new structures shall be in accordance with AS 5100, Bridge Design (CODE). There are additional MRWA requirements, and these are described below. Where there is any conflict between the requirements of the CODE and the following, the latter shall take precedence.

Detailed guidelines applicable for vehicle loading and positioning are given in Branch Design Information Manual (BBDIM) Document No. 3912/02-04 “Load Rating Bridges”. Please note, the full width between barriers on the bridge shall be considered for design vehicle loadings to allow for any future changes ignoring internal kerbs, footpaths, traffic barriers, median barriers, and medians.

For load rating of bridges, the CODE, Document No. 3912/02-04 “Load Rating Bridges” and Document No. 6706-02-2227 “Load Rating and Refurbishment Design Manual for Existing Timber Bridges” are applicable.

## 5.2 Special Vehicle Loading

Structures located on highways, main roads and designated Heavy Haulage Routes shall be designed to Supervised Group 2 Vehicle 4, Group 2 Vehicle 5 and HLP 400 heavy load platform vehicles unless EBL specifies otherwise.

Structures that are part of a shared path where access by a maintenance, inspection or emergency vehicle is possible must be designed for a “Service Vehicle”. The configuration of the “Service Vehicle” is the same as an M10 truck (M-Truck), defined in Figure A8 of AS 5100.7-2004 with total mass of truck M = 100 kN. For performing load rating, the “Service Vehicle” shall be modelled and positioned similar as T44 vehicle (refer to Document 3912/02-04). The Load Factor and the Dynamic Load Allowance (DLA) shall in accordance with AS5100.2 defined for pedestrian, cyclist path and maintenance traffic loads. Structures protected by bollards are not exempt from this requirement, however shared path bridges containing loop ramps at each end are not required to comply.

## 5.3 Oversize and Over-Mass Vehicles

The design criteria for Oversize and Over-Mass load special vehicles shall be applied as additional specific load cases for all structures located on designated Oversize and Over-Mass Vehicle Corridors. Details of these special vehicles, designated routes and design guidelines are located online at www.mainroads.wa.gov.au, Guide to Design of Oversize and Over-Mass Vehicle Corridors.

The application of Oversize and Over-Mass vehicles for the design of new structures shall be confirmed with the Design Criteria Sheet and approved by SES.

## 5.4 Reduction of Standard Highway Loading

The use of less than full loading as outlined above on structures should only be considered in special cases as it is often a false economy and removes an important safety factor against overloading. Each instance must be approved individually by SES.

**SECTION 6 – STRESS LIMITS IN STRUCTURAL CONCRETE**

This information is Part 6 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 10/05/18

**Document No: 3912/02-6**

**SECTION 6**

**CONTENTS**



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 12/12/05 | Complete review for introduction of AS 5100 |
| All | 2 | 04/04/18 | Complete review for introduction of AS5100-2017 |
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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 10/05/18

# 6 STRESS LIMITS IN STRUCTURAL CONCRETE

## 6.1 PRESTRESSED CONCRETE

When carrying out prestressed concrete design one of the Serviceability Limit State checks is for crack control in flexure. This is covered in AS 5100, Bridge Design (CODE), Part 5, Clauses 8.6.2 and 9.4.2 for the design of new bridges or load rating of bridges constructed using 500MPa steel reinforcement.

Additional requirements for prestressed concrete design limits on steel stress at the serviceability limit state are given in Table 6.1 for load rating of bridges constructed using different steel grades, and are applicable to both superstructure and substructure design. The values below are the limit on the increment in reinforcement bar stress between decompression and the applicable SLS load combination.

Table 6.1

| Nominal Steel Grade (MPa) | Serviceability Stress Limits for Prestressed Concrete | | |
|---|---|---|---|
| | T44/L44 & Road Trains | M1600 | HLP & Special Vehicles |
| 200 | 80 | 90 | 100 |
| 230 | 90 | 105 | 115 |
| 400 | 160 | 180 | 200 |

## 6.2 REINFORCED CONCRETE

Reinforced concrete requires different limitations on stress for serviceability compared to partially prestressed concrete due to the lower vulnerability of reinforcement to fatigue and corrosion.

When carrying out reinforced concrete design one of the Serviceability Limit State checks is for crack control in tension and flexure. This is covered in the CODE, Part 5, Clauses 8.6.1 and 9.4.1 for the design of new bridges or load rating of bridges constructed using 500MPa steel reinforcement.

Additional requirements for reinforced concrete design limits on steel stress at the serviceability limit state are given in Table 6.2 for load rating of bridges constructed using different steel grades, and are applicable to both superstructure and substructure design.

Table 6.2

| Nominal Steel Grade (MPa) | Serviceability Stress Limits for Reinforced Concrete | | |
|---|---|---|---|
| | T44/L44 & Road Trains | M1600 | HLP & Special Vehicles |
| 200 | 120 | 135 | 150 |
| 230 | 125 | 140 | 160 |
| 400 | 180 | 200 | 220 |

**SECTION 7 – RAILINGS AND BARRIERS**

This information is Part 7 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 23/08/24

**Document No: 3912/02-7**

**SECTION 7**

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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 23/08/24

# 7 RAILINGS AND BARRIERS

## 7.1 Traffic Barriers

### 7.1.1 General

Traffic barriers are all barriers attached to a structure including bridges, underpasses, tunnels, culverts, retaining walls, traffic barrier footings, and any associated approach slabs.

For maintenance reasons it is preferred to use a barrier from the Main Roads standard drawings where possible.

The same traffic barrier system must be used on both deck edges where possible. An accepted reason to use differing barriers is to have a suitable height barrier on one side for pedestrian/cyclist safety but a lower height barrier on the opposite deck edge to permit High Wide Load movements.

Post holding down bolts must be designed such that they can be easily replaced if damaged during a crash.

The setback distance from the kerb face to the traffic barrier and height of rail above the road surface must be in accordance with the Main Roads standard drawings unless otherwise agreed to and approved in writing by Senior Engineer Structures (SES).

The Code does not allow design of a barrier system by calculation, only design of prototypes for crash testing and design of modifications. It is important to note that crash testing or in-service performance evaluation to AS/NZS 3845.1-2015 is required to confirm barrier performance. Since the 2015 publication of AS/NZS 3845.1 the traffic barrier performance standard has been updated from NCHRP-350 to MASH.

### 7.1.2 Barrier Selection

The required barrier performance level shall be assessed in accordance with the Code. Where assessment determines that regular performance level barriers are appropriate:

1. In situations where there is not a significant risk of secondary incidents due to the land use adjacent or below, steel AS 5100-2004 compliant barriers are permitted.
2. In other instances, to achieve MASH compliance, the barrier shall be the steel/steel & concrete hybrid TL5 barrier or concrete TL4 barrier.
3. Other proprietary MASH compliant barriers are acceptable on a case-by-case basis where approved by SES. Adequate justification must be submitted.

However, barrier selection is not based purely on the performance level assessment from AS5100.1. The engineer must also consider factors such as:

- The impact severity, which is worse for rigid barriers.
- How long it might take to repair a barrier so that it is effective against successive impacts.
- The presence of screening or lighting, which must remain outside zones of intrusion (see Section 7.5 for definition of zone of intrusion)
- Regional preferences

### 7.1.3 Steel/Concrete Hybrid Barrier

In 2023, a steel/concrete hybrid TL5 guardrail was successfully crash tested as a MASH TL5 barrier [1]. The tested barrier configuration was the Type MAO barrier as per TfNSW drawing B0505. Main Roads Structures Engineering is allowing the barrier to be implemented in advance of the Austroads full crash test report and preparation of corresponding Main Roads WA Standard Drawings.

The steel/concrete hybrid barrier may be accepted as an edge barrier for scenarios where cyclists could access the bridge. The engineer must provide justification as to why other barrier types of sufficient height were not suitable.

### 7.1.4 Steel Barriers

Main Roads does not have a public domain regular performance MASH compliant steel barrier as a standard; however, a medium performance system is available.

Where the Main Roads medium performance steel MASH compliant traffic barrier is being considered, note that:

- Code requirements for cyclists apply. In effect, the barrier is not suitable as an edge-of-deck barrier when cyclists may be adjacent. In this instance, the steel/concrete hybrid barrier can be considered. Alternatively, it is currently permitted to install an AS5100-2004 compliant barrier where the required performance level of the barrier is assessed to be regular.
- Post spacings are 1.22m. The bridge length often needs to be increased to ensure that posts are not too close to the edge of concrete at the bridge expansion joints. This is to prevent concrete blowout under collision loading.

To facilitate future upgrades, the structure and guardrail connection shall always be designed for MASH compliant loads regardless of whether the barrier is compliant:

1. Deck reinforcement design should assume the same post spacing as the relevant AS 5100-2004 barrier. However, the design loads from AS5100-2017 apply.
2. Line of sight implications for AS5100-2017 barrier geometry (refer table 12.2.3 of AS 5100.2) shall be checked with outcomes recorded in the design report & drawings.
3. The design summary, typically located on the General Arrangement drawing, shall note:
    a. Barriers: AS5100-2004
    b. Barrier Anchorage and bridge deck: AS5100-2017

Low performance barriers for cyclists such as heritage rail with top-rail or low performance Thriebeam (side-mounted PFC posts and PFC blockouts) with top-rail require SES approval. Note that top-rail on a UC post is not a tested configuration and therefore not accepted.

In general, to avoid re-design of the bridge for an increased span, for steel post and rail barriers, the barrier post set-out shall be confirmed before detailed structural design commences. Posts shall be positioned at a sufficient distance from expansion joints and the edge of concrete to prevent failure in the event of a collision.

### 7.1.5 Concrete Barriers

For concrete barriers:

1. The preferred concrete barrier is the constant slope type.
2. Where the barrier does not meet the offsets in Main Roads Supplement to Austroads Guide to Road Design Part 6 Clause 6.3.5, the F Type is preferred.
3. Top-rail is not acceptable on concrete barriers without SES approval. The standard steel/concrete hybrid barrier shall be considered instead.

Concrete traffic barriers on bridges must not be considered as part of the deck and therefore must not be considered as contributing to its strength.

Where a concrete barrier has a cavity, for example to house lighting control gear, the cavity must not create a snag hazard under traffic impact. For example, the access shall be located on the footpath side where available, elsewise a sufficiently thick steel cover plate may be required.

### 7.1.6 Transitions and Extents

Start and end points of barriers must comply with 'length of need' requirements and extend adequately off the bridge to protect motorists from hazards in accordance with Austroads Guide to Road Design Part 6 and the associated Main Roads supplement. Examples of hazards include vertical drop, steep embankment, road or highway, railway, hazards in the median, or any combination of these.

Where access prevents installation of the required length of need or preferred barrier then it may be appropriate to alter the barrier design. However:

1. It must also be demonstrated that vehicles, either on the main alignment or intersecting road, are adequately shielded from high severity roadside hazards, e.g., the barrier may need to continue around intersecting roads to protect traffic on the main alignment.
2. Crash history shall be checked to ensure that curtailing the barrier does not cause incidents to repeat.
3. Road design sight distances shall be investigated.
4. In some instances, it will be required to liaise with the property owner to consider moving the access paths to create an outcome that minimises safety issues.
5. Access for inspection and maintenance purposes shall also be considered.

Transitions from one barrier type to another must be done such that the performance of either barrier is not compromised, no unsafe ends are exposed, and the appearance is neat. To avoid sudden changes in stiffness, short lengths of barrier joining onto a longer length of a different type of barrier must be avoided.

The ends of all approach and departure barriers must be terminated with a crashworthy end terminal in accordance with Austroads Guide to Road Design Part 6 and the associated Main Roads Supplement.

Any new structure or refurbishment omitting guardrail must justify the decision, considering factors such as but not limited to whether existing guardrail is present, whether there is a local crash history, local road hazards and road geometry, the type of vehicles using/expected to use the crossing, AADT, etc. Formal approval from SES must be sought for scenarios where the omission of guardrails (including omitting guardrail extensions off bridge) is proposed.

## 7.2 Balustrades

Balustrades on road bridges, and on underpass headwalls and wingwalls where the wall is within 3m in plan of the edge of the path, must be standard Main Roads rectangular hollow section balustrade with solid balusters.

Balustrades on shared path bridges and associated ramps must comprise circular rails and circular balusters spaced in accordance with Code requirements.

Balustrades other than Main Roads standard are permitted where:

(1) The structure is of significant aesthetic importance.

(2) Code requirements are met (AS 5100, taking priority over AS 1428.1)

(3) The alternative balustrade is commended by the project Architect.

(4) The design life can be achieved with no maintenance other than painting.

(5) The design allows convenient repair/replacement of all components.

(6) AMS or Structures Engineering have not raised concerns.

(7) It is expected that existing suitable designs shall be used if they exist, with modifications for any identified problems.

The standard Main Roads balustrade must extend no less than 3 complete panel lengths off the bridge from the abutment bearing centreline, and for bridges with abutment wingwalls adjacent to shared paths or footpaths the balustrade must extend at least 3m from the end of the wingwalls. A different type of barrier separating path users from hazards can be used beyond this point.

It should be noted that the standard Main Roads balustrade was not developed for crowd loading.

Regardless of adjacent civil path design, all bridge balustrades shall allow for cyclists. Main Roads has investigated Gooseneck balustrade. It has been agreed with WestCycle that Gooseneck balustrade shall not be used on bridges (refer D24#987456).

The designer shall be considerate of detailing that allows ingress of water to tight spaces, eg: between edge-mounted steel plates and/or the deck. The retention of water may cause localised durability issues, such as rusting of anchor rods and consequently cracking & spalling of the deck well in advance of the design life. Note that silicone for water proofing is considered high maintenance and difficult to fund, therefore it should be avoided when possible. This is because the silicone perishes or hardens in a relatively short timeframe and if the bridge does not have other specific maintenance items, then the silicone maintenance becomes a relatively low priority for maintenance funding. This is an especially common scenario for footbridges.

## 7.3 Railway Protection Screens

Electrification protection screens must be provided to bridges over the electrified railway in accordance with PTA Specification 8880-450-061 Protection Screens, and Main Roads requirements. Furthermore:

- The Memorandum of Understanding between Main Roads and the Public Transport Authority defines the ownership of Railway Protection Screens, refer D11#290707. In essence, Railway Protection Screens are owned by Main Roads if they are integral with balustrade or bridge barriers. All other Railway Protection Screens are owned by the PTA. Note that Railway Protection Screens should not be confused with screens used for privacy and other purposes, even though all screens often appear the same for aesthetics.
- Regardless of ownership, Main Roads requirement for Railway Protection Screens is that they are located outside the Zone of Intrusion (see Section 7.5 for definition of Zone of Intrusion)
- The minimum clearance requirements of both PTA and Main Roads must be met
- Screens must be provided on both sides of each structure and must extend a minimum of 5.0 m beyond the centreline of the railway line
- Carefully consider painting as it may inhibit graffiti removal. Liaise with the PTA and architect on screen and finish type

## 7.4 Anti-Throw Screens

Where Anti-Throw screens and Railway Protection Screens are both required, Anti-Throw screens must be able to fulfil the function of Railway Protection Screens.

As a minimum, the Main Roads process “Risk Assessment for Projectiles Thrown from Overpass Structures” (D23#845813) shall be used to determine the need for screening. Risks other than projectiles may need to be considered using a general risk assessment procedure.

In instances where screens are not part of the project scope, and the structure is over a freeway or highway then the design shall allow for future retrofit of screens without requiring strengthening works unless otherwise agreed with and approved by Senior Engineer Structures.

## 7.5 Zone of Intrusion

Zones of Intrusion for <u>rigid</u> barriers on bridges shall be as per Table 7.1.

The Zone of Intrusion (ZOI) is defined as the distance between the top roadside corner of the barrier before an impact and the maximum lateral position of any part of the vehicle during and after an impact (measured outwards from the traffic side of the barrier)

The values in Table 7.1 are from physical crash tests so far as possible. Interpolation or estimation of alternative values is strictly not permitted. Details of the crash tests referred to above can be found in [1], [2] and [3]. For a given rigid barrier height, where no existing test data was found for a particular test level, the lesser of the following was adopted as the applicable ZOI;

- ZOI of the closest shorter barrier tested to the same test level
- ZOI of an equal height barrier tested to a higher test level

It should be noted that a ZOI equal to 830mm was observed for the TL5 crash test carried out for the TfNSW Type MAO barrier [1].

| Rigid Barrier Test Level and Height (mm) | | Zone of Intrusion (mm) | |
|---|---|---|---|
| | | Cab | Cargo Box |
| TL-4 | 915 | 864* | 2032* |
| | 1070 | 610* | 2032 |
| | 1400 | 457 | 830 |
| | 1420 | 457 | 830 |
| TL-5 | 1070 | 1220* | 2200* |
| | 1370 | 457* | 1140* |
| | 1400 | 457 | 830* |
| | 1420 | 457 | 830 |

Table 7.1: Zones of Intrusion on and under Structures (*actual crash test data)

Important notes on usage:

- It is not possible to visually delineate concrete TL4 barriers from TL5 barriers if they are the same height. Hence Structures Engineering requires all concrete barriers 1070mm or greater in height to meet TL5 performance requirements. The structure and connection to it must be designed accordingly.
- Special cases exist for the upgrade of barriers on existing bridges where the Zone of Intrusion must be limited but the bridge cannot accommodate TL5 loading. Hence Table 7.1 contains information on TL4 barriers of 1070mm or greater height. Written approval must be obtained from Senior Engineer Structures to implement a TL4 barrier of these heights.
- The ZOI values in Table 7.1 were developed in conjunction with the Road & Traffic Engineering Branch.
- For TL4 rigid barriers of 820mm height use the ZOI values in Table 7.1 for a 915mm high TL4 barrier.
- Height of the barrier is measured from the adjacent road surface.

## 7.6 Standards and Approval

Refer to Structures Engineering standard barrier and balustrade drawings in the Standard Drawings section of the Main Roads website. Refer also to the 'List of Approved Road Safety Barrier Systems'.

The omission of barriers or reduction in length of need will not be supported by Structures Engineering without detailed justification in the design report.

## 7.7 References

[1] Holmes Solutions, 2023, NTRO on behalf of Austroads Compliance Testing of the TfNSW Medium Performance Traffic Barrier Type MAO Modified MASH TL5, Version 1.2 (draft version published for internal distribution only)

[2] Keller, E.A., Sicking, D.L., Faller, R.K. and Polivka, K.A., 2003. *Guidelines for Attachments to Bridge Rails and Median Barriers,* Mid-America Transportation Center, University of Nebraska-Lincoln.

[3] Hobbs, S.F., 2010. *Zone of Intrusion and Concrete Barrier Countermeasures*. In Annual Conference of the Transportation Association of Canada.

# SECTION 8 – BEARINGS AND JOINTS

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## Authorisation

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**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 04/09/2023

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**SECTION 8**

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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 04/09/2023

# 8 BEARINGS AND JOINTS

## 8.1 Bearing Design

The design of bearings is covered extensively in Part 4 of AS 5100, Bridge Design (CODE). Chapter 16 of the Bridge Branch Design Manual Document 3912/03 also contains useful information.

*Elastomeric Bearings*

Design Summary Notes, usually presented on the General Arrangement drawings, shall indicate the expected movement ranges at suitable timeframes (construction, 7 years, 15 years, 30 years) so that it can be easily determined whether measured deflections are within the design range. This ensures that bearings that are operating within specification are not mistaken as defective and flagged for maintenance. This has been an ongoing problem for long span bridges where creep and shrinkage are significant.

The ability to replace and re-set bearings is required as part of the bridge design. This will typically require structural design and geometric allowance for temporary jack locations and summarising pertinent information (eg: allowable jacking sequences and limitations on differential & total jacking).

Elastomeric bearings require keeper plates top and bottom (refer Section 2), regardless of whether minimum compression loads are met as per the Code. The designer is to allow for the reduced effective height of the bearing due to the keeper plates. Bolted plates are preferred over welded plates. Plates that are welded on a single side cannot provide a moment resisting couple and are therefore not allowed.

*Other bearing types*

There can be problems with overstressing pot bearings – particularly with a rotation movement under pressure. There is a ring underneath the elastomer that prevents egress down the piston when the elastomer is compressed. The ring can fail, which is particularly common for bearings that have the ends of the ring welded together. Consequently the elastomer can flow through the gap that is created and be permanently damaged.

## 8.2 Expansion Joints

Some form of expansion joint is required on most bridges to cater for longitudinal movements due to temperature change, concrete shrinkage and concrete creep. In fact one of the first decisions in a bridge design is what restraint system is to be adopted, i.e., where is the structure to be fixed and therefore where and in what direction is movement permitted.

The magnitude of anticipated movements due to temperature, creep and shrinkage can be calculated from figures given in the Code. Temperature movement can be +ve or -ve (expansion or contraction), whereas creep and shrinkage are only -ve (contraction). It is essential to ensure that there is adequate capacity for movement over the full temperature range, both early in the life of the bridge before much creep and shrinkage has occurred and ultimately after full creep and shrinkage. Transverse movements need also be considered.

The concrete surrounding the expansion joint shall incorporate an upstand to suit the following seal types. The following table indicates typical values that should be confirmed with pavement design requirements for final design.

| Seal Type | Required Concrete Upstand |
|---|---|
| 2-coat seal (typical rural treatment) | 20 mm |
| Asphalt / Hot-mix (rural) | 40 mm |
| Asphalt / Hot mix (urban) | 50 mm |

The options available for expansion joints vary according to the anticipated range of movement and the type of structure, and reference to the Bridge Branch Design Manual Document 3912/03 should be made for details of the various types of joint and their applications. Chapter 16 of 3912/03 also gives a guide to the selection of an appropriate expansion joints for different bridge lengths.

Where finger plate joints are used, the fingers are to remain simply supported by extending across the joint. Cantilevered fingers have failed due to overstress or fatigue.

Modular joints must have a noise attenuation system if used in an urban area. The noise attenuation system must be supplied by the joint manufacturer and cannot be retrofitted with welded surface mounted plates. The top of the joint, including any noise attenuation system, must be at road surface level. Sufficient space must be provided under the Deck joint for access by maintenance personnel to safely remove and replace all joint components.

Advice on common types of expansion joints is provided below. Note that regional preferences may exist.

- Elastic polymer plug joints have been installed on Bridge 1009 Mill Point Road over Kwinana Freeway in 2016. At this time they are not considered suitable for high traffic volumes. This joint has also been installed on Bridges 87, 228, 270A, 572A, 1272, 5370.
- Poured flexible sealant has been used on multiple bridges, either between existing expansion joint angles or with new polymer nosing. The performance is extremely dependent on the quality of the installation. A good bond is often not achieved between existing expansion joint angles and the joint sealant. No comment is available on suitability when used with polymer nosings.
- Precompressed silicone and foam hybrid is currently being trialled, initial impressions of waterproofing are positive. These products are not suitable for installation between metal angles in any climate as the foam rapidly deteriorates. A construction drawback is that some products are supplied in 2m segment lengths, requiring numerous joints with joining silicone.

Where accessible by cyclists, anti-slip surfaces and gap widths between teeth (etc) must be suitable.

- For new bridges, cyclist requirements must be met without requiring modification of the proposed product.
- Where anti-slip plates have been welded to existing joints it has been found that the plates have dislodged and have the potential to become missiles.

Furthermore there is question as to whether the underlying joints have been damaged due to continued re-welding of the anti-slip plates. This and the ongoing maintenance needs to be considered versus the safety benefits.

## 8.3 Approach Slabs

Approach slabs are required for several reasons, including mitigation of settlement of approach embankment fill, provision of anchorage for expansion joints and approach transitions to reduce impact and sudden bumps.

Approach slabs shall be sufficiently thick to provide adequate support and anchorage for expansion joint angles and guardrail post anchor bolts, with consideration given to the ability of the slab to span over potential hollows or voids which may occur beneath the slab due to settlement or wash-out. Consideration must also be given to drainage and provision of kerbs etc.

If the road embankment is supported by solid rock and the abutment foundation is a spread footing on solid rock, then the approach slab shall be designed as a slab on elastic foundations. Controlled fill embankments with no possibility of scour or large settlements shall also be designed in this manner.

However, if underlying soils are soft and have the propensity for long term settlements then the approach slab shall be designed as simply supported, to span from the abutment support to the centreline of the trailing edge support of the approach slab.

Approach slabs designed in conjunction with timber bridge concrete overlays shall be in accordance with Document No. 6706-02-223, Structures Engineering Practice Notes.

At some sites it may be too disruptive to excavate deep enough to construct a corbel with an on-ground approach slab. In such cases an approach beam may be appropriate. In general an approach beam is only suitable for existing structures in the 20-30m length range where there will be limited settlement. It is not appropriate for situations where there may be scour.

# SECTION 9 – CONSTRUCTION FORCES AND EFFECTS

This information is Part 9 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

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**Authorisation**

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SENIOR ENGINEER STRUCTURES

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**SECTION 9**

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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 04/09/2023

# 9 CONSTRUCTION FORCES AND EFFECTS

This section is no longer used:

- For new bridges, the requirements have been transferred to Section 2.
- For maintenance and refurbishment, Section 3 requires the engineer to meet the same requirements as for new bridges.

Section 9 will be deleted in a future revision.

# SECTION 10 – CONCRETE STRENGTHS AND FINISHES

This information is Part 10 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

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**A LIM**
SENIOR ENGINEER STRUCTURES

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**SECTION 10**

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Custodian Endorsement

M RAJAKARUNA
Structures Design and Standards Engineer
Date: 13/04/18

## 10 CONCRETE STRENGTHS AND FINISHES

### 10.1 CONCRETE STRENGTHS

Unless otherwise approved by the SES, the following standard concrete strengths shall be used in the design of new bridges and the refurbishment design of existing bridges:

| Concrete Strength | Class | Comments |
|---|---|---|
| 20MPa | N20 | Non-structural works only where durability is not an important factor, e.g., – blinding, post footings, mass concrete fill, paths etc |
| 35MPa | S35/10 | Standard reinforced concrete for precast parapet panels |
| 40MPa | S40 | Standard reinforced concrete and prestressed concrete, e.g., – foundations, piers, decks, concrete overlays, beams etc |
| 50MPa | S50 | Precast reinforced concrete, precast prestressed concrete, precast piles, beams, stress anchorages etc |
| | S50M | Reinforced concrete substructures in marine applications. |
| 65MPa | S65 | Precast prestressed beams |

The 20 MPa is a standard AS 3600 mix and is designated N20. All others are specifically defined mixes as per MRWA Specification 820.

Although higher concrete strengths are included in the Bridge Code, parameters for higher strength mixes do not currently form part of Main Roads Specification 820. Higher strength mixes and/or special concretes can be used in special situations, e.g., prestressed concrete footbridges, precast work, high early strength, self-compacting concrete, sulphate resisting etc. The use of higher strength and/or special concretes is to be confirmed in the Design Criteria Sheet and approved by the SES.

### 10.2 FINISHES

All concrete surfaces should have the standard of finish indicated on the Drawings. There are two broad groups of concrete finishes – formed and unformed surfaces.

Formed Surface Finishes - shall comply with the requirements of Section 3 of AS 3610 - "Formwork for Concrete". Finishes Type 2, 3 and 4 cover most situations for bridge works and Table 10.1 gives guidance as to where they are to be used. Refer to the Standard for full details of allowable tolerances, colour variation, etc for each finish. Special formed finishes may also be used, e.g. ribbed, rope, board marked etc, but the above three cover the majority of cases.

Unformed Surface Finishes - four classes, U1 to U4 as per Table 10.2, cover all normal requirements. Note that the finishes and irregularities do not scale from best to worst as per formed finishes. The user is referred to the “typical location” in Table 10.2 to ensure appropriate unformed finishes are selected.

## 10.3 PROPRIETARY MORTAR AND REPAIR GROUTS

Proprietary mortars and grouts are often used for bearing pads, concrete repairs, infill concrete and other special applications. They shall be selected based on the appropriate requirements for strength, workability, performance or other special characteristics from well-known suppliers and/or manufacturers.

In general, such proprietary products should be readily available within Western Australia or Australia, and supported with detailed Technical and Safety Data Sheets.

Where a particular product has not been used previously on a MRWA project, it shall be approved by the SES prior to use in the Works. Consultants proposing the use of new products shall seek approval for its use prior to finalising the design or Works commencing.

## TABLE 10.1 – FORMED SURFACE FINISHES FOR CONCRETE

| Designated Finish | Typical Location | Type of Concrete Finish[1] | Maximum Allowable Surface Irregularities |
|---|---|---|---|
| 2 | Exposed surfaces, general external and internal surfaces intended to be viewed in detail | Smooth, dense and dust free concrete finish uniform in colour and accurately formed to specified dimensions and tolerances. Joint marks to be unobtrusive and concrete surfaces to be free from air holes and effects of water migration. Panels to be arranged in an approved regular pattern conforming to the structural geometry. | 3 mm abrupt or 6 mm in a 1.5m template |
| 2X | As specified on the Drawings | Sandblasted concrete finish, or special architectural applications, otherwise as per Type 2 above. | As specified |
| 3 | General external or internal surfaces intended to be viewed as a whole, or unexposed surfaces hidden from view | Regular, dense and dust free concrete surface entirely free from honeycombing and effects of cement paste leakage. | 5 mm abrupt or 7 mm in a 1.5m template |
| 4 | Footings and buried surfaces | Structurally sound and durable concrete with a dense surface free from honeycombing. | 8 mm abrupt or 10 mm in a 1.5m template |

Note: Table 10.1 provides a general description only. Refer to AS 3610 for detailed descriptions and classifications.

## TABLE 10.2 – UNFORMED SURFACE FINISHES FOR CONCRETE

| Designated Finish | Typical Location | Type of Concrete Finish | Maximum Allowable Surface Irregularities |
|---|---|---|---|
| U1 | Unexposed surfaces | A wood floated finish to produce a uniform, dense concrete surface free of surface pitting or cavities. | 5 mm abrupt or 15 mm in a 3m template |
| U2 | Upper exposed surfaces | A high quality steel trowelled finish to produce a uniform, dense, smooth and impervious concrete surface finish free of surface pitting or cavities. | Nil abrupt or 5 mm in a 3m template |
| U3 | Upper surfaces of bridge decks and approach slabs/spans | A high quality mechanical steel trowelled finish to produce a uniform, dense impervious concrete surface finish free of surface pitting or cavities. | 2 mm abrupt or 5 mm in a 3m template |
| U4 | Upper surfaces of bridge deck concrete overlays, footpaths and medians | A high quality wood floated finish to produce a uniform surface free of surface pitting or cavities followed by a broom finish to produce a uniformly roughened concrete surface free of excessive drag marks and overlaps. | 2 mm abrupt or 5 mm in a 3m template |

**SECTION 11 – BRIDGE WIDTHS**

This information is Part 11 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 19/03/25

**Document No: 3912/02-11**

**SECTION 11**

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 13/12/05 | Complete review for introduction of AS 5100-2004 |
| 4 | 2 | 13/11/14 | Updated Principal Design Engineer position in R&TE branch. |
| 5 | 2 | 13/11/14 | Table 11 - revised width requirements for roads other than National Highways |
| All | 3 | 19/07/17 | Complete review against AS5100.1-2017. No changes required. |
| 3-4 | 3 | 19/07/17 | Asset Owner, endorsements and approvals updated. |
| 5 | 3 | 19/07/17 | "Sealed Shoulder Width" amended to "Total Sealed Shoulder Width" and notes adjusted accordingly. |
| 6 | 3 | 19/07/17 | "Shoulder Width" amended to "Total Shoulder Width". "Location" fields updated. Endorsements and approvals amended. |
| 3 | 4 | 21/09/17 | Endorsements and approvals amended |
| 5 | 5 | 21/05/18 | Table 11 AADT ranges updated. Rural Bridge Width Approval Form revision number amended. |
| 4 | 6 | 19/03/25 | Minimum bridge width for Main Roads and Highways amended |

Custodian Endorsement

**M RAJAKARUNA**
Structures Design & Standards Engineer
Date: 19/03/25

# 11 BRIDGE WIDTH APPROVAL PROCEDURE

This Section covers the procedure to be adopted for the determination and approval of bridge widths. It is split into two parts, Urban and Rural, as the criteria for assessing each are different. Rural bridge widths are standardised and determined based mainly on anticipated traffic flows. Urban bridge widths are subject to more variation.

The definition of 'urban' and 'rural' in this context is fairly subjective, being based more on the type of road/structure than its geographic location. The Structures Design & Standards Engineer shall be responsible for deciding which part of the procedure to follow.

In all cases, consideration should be given to the possible need for future widening.

## 11.1 URBAN BRIDGES

The Designer shall seek advice from the Principal Design Engineer, Road & Traffic Engineering Branch, on the required bridge width, including carriageway widths, shoulders, medians, dual use paths etc.

These requirements shall be incorporated into a schematic bridge cross-section, placed on the Bridge Design Part File and circulated for approval as per Paragraphs 11.3 and 11.4 below.

## 11.2 RURAL BRIDGES

For each rural bridge, the Designer shall place a Rural Bridge Width Approval Form 3912/02/11/01 and a copy of the best available map on file for the relevant Region. The map should show the location of the bridge site including details of the road and the Local Government area in which it is located.

Bridge width requirements are described in Section 11.6 below.

## 11.3 ASSET OWNERS ADVICE

The Designer shall obtain advice from the Asset Owner as to local requirements for bridge width. For MRWA owned bridges, the Asset Owner is the regional Asset Manager Structures.

## 11.4 ENDORSEMENTS AND APPROVALS

The schematic bridge cross-section (in the case of urban bridges) or the completed Rural Bridge Width Approval Form (in the case of rural bridges) shall be sent to:

- Asset Owner — for advice
- Principal Design Engineer — for advice (urban bridges only)
- regional Asset Manager Structures — for endorsement

Following endorsement, forwarded to:

- Senior Engineer Structures — for authorisation

Following authorisation, the regional Asset Manager Structures is responsible for placing the signed Rural Bridge Width Approval Form on the specific bridge design or maintenance file as appropriate.

## 11.5 RESPONSIBILITIES FOR CONSULTANTS

When the design is undertaken by Consultants, then unless otherwise specified in the consultants brief, the Consultant shall assume all the responsibilities of the Designer in the above procedure except that the MRWA Project Manager shall arrange for filing and returning a copy to the Consultant after actions by Structures Engineering as detailed above.

## 11.6 BRIDGE WIDTH REQUIREMENTS

The recommended minimum rural road and bridge width between kerbs for Roads other than Main Roads and Highways shall be in accordance with Table 11.

The minimum bridge width between kerbs on Main Roads and Highways shall be the Ultimate (2040+) Pavement Width given in the Main Roads Supplement to the Austroads Guide to Road Design Part 3: Geometric Design, section 4.1A [refer to Ultimate (2040+) Cross Section].

For any curve widening width requirements, refer to Section 7.9 of Austroads Guide to Road Design: Part 3 the Geometric Design,

These width requirements do not apply to existing bridge refurbishment and widening work.

## TABLE 11 RECOMMENDED MINIMUM RURAL ROAD AND BRIDGE WIDTHS

(Roads other than Main Roads and Highways)

| | UNSEALED ROADS | SEALED ROADS | | | | |
|---|---|---|---|---|---|---|
| | | SINGLE LANE | TWO LANES | | | |
| DESIGN AADT (VPD) | <100 | <150 | 150-500 | 500-1000 | 1000-2000 | 2000-8000 |
| Sealed Trafficway Width | Not Applicable | 3.5 | 6.0 | 7.0 | 7.0 | 7.0 |
| Total Sealed Shoulder Width (1) | Not Applicable | 1.5-2.5 | 1.0-1.5 | 1.5-2.0 | 2.0 | 2.0 |
| Formation Width | As Appropriate | 6.5-8.5 | 9.0 | 10.0 | 11.0 | 12.0 |
| Bridge Width Between Kerbs (2) | 4.2 (3, 4) | 4.2 (3, 4) | 7.2 (4) | 8.2 (5) | 8.2 (5) | 8.2 (6) |

Abbreviations: AADT – Annual Average Daily Traffic
VPD – Vehicles Per day

NOTES:

1. The Total Sealed Shoulder Width is the sum of both shoulders
2. Where there is a kerbed footway on the bridge, the kerb shall be set back a minimum of 600mm from the edge of the adjacent traffic lane
3. Where sight distance is inadequate or bridge length is less than 10m, the minimum width shall be 7.2m
4. For length of bridge 6m or less, the width shall be full formation width
5. For length of bridge 9m or less, the width shall be full formation width
6. For length of bridge 15m or less, the width shall be full formation width

References: AS 5100-2017, Bridge Design, Part 1
AUSTROADS Guide to Road Design, Part 3: Geometric Design

# RURAL BRIDGE WIDTH APPROVAL FORM

**3912/02/11/02**

**LOCATION**

BRIDGE NUMBER ______________ ROAD NUMBER ______________ ROAD NAME ______________

BRIDGE OVER ______________________________ AT SLK ______________

ASSET OWNER ______________ REGION ______________ LOCAL GOV ______________

**PROPOSED STRUCTURE**

TYPE OF BRIDGE ______________________________

NUMBER OF SPANS ______________ TOTAL LENGTH ______________

**TRAFFIC DATA**

ACTUAL AADT (vpd) ______________ (Year) ______________

DESIGN AADT (vpd) ______________ (Year) ______________ ______________ DESIGN SPEED (km / hr)

| | TRAFFICWAY WIDTH (m) | CURVE WIDENINGS (m) (IF REQUIRED) | TOTAL SHOULDER WIDTH (m) | | FORMATION WIDTH (m) | BRIDGE WIDTH BETWEEN KERBS (m) |
|---|---|---|---|---|---|---|
| | | | SEALED | TOTAL | | |
| Existing | | | | | | |
| Minimum (1) | | | | | | |
| Proposed | | | | | | |

(1) From Table 11 ______________________________

Comments: ______________________________

______________________________

______________________________

______________________________

Recommended By ______________________________ Date ____ / ____ / ____

DESIGN ENGINEER

Endorsed By ______________________________ Date ____ / ____ / ____

DESIGN SECTION LEADER

**Asset Owner Endorsement**

______________________________

Advice from Asset Owner at folio
(N/A for MRWA Asset Owners) ______________

If approved width different to Asset Owners advice resubmit for agreement
Asset Owner signature required prior to submitting to SES for authorisation

**SES Authorisation**

______________________________

______________________________

______________________________

# SECTION 12 – CLEARANCES AND HIGH LOAD ROUTES

This information is Part 12 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Asset Policy Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 28/08/18

**Document No: 3912/02-12**

**SECTION 12**

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| 2 | 1 | 04/03/04 | Clause 12.2 amended |
| 5 | 1 | 04/03/04 | Figure 12.1 amended |
| All | 2 | 13/12/05 | Complete review for introduction of AS 5100-2004 |
| 3,4 | 1 | 24/8/07 | Figure Nos changed |
| 6,7 | 1 | 24/8/07 | Figure 12.2 drawing updated |
| 8 | 1 | 24/8/07 | Figure 12.4 drawing added |
| 9 to 11 | 1 | 24/8/07 | Figures 12.5 to 12.7 changed. |
| 5 | 2 | 11/01/12 | Figure 12.1 amended |
| All | 3 | 20/08/18 | Complete review for introduction of AS5100-2017 |
| | | | |
| | | | |
| | | | |

Custodian Endorsement

J PARVIN
Structures Asset Policy Engineer
Date: 24/08/18

# 12 DESIGN VEHICLE LOADINGS

## 12.1 Introduction

This Section sets out the minimum clearances, both horizontal and vertical, to be used in the design of bridges. It generally follows the requirements of Part 1, Clause 13 of AS 5100, Bridge Design (CODE), but where there is any conflict this document shall take precedence. Where none of the following considerations are applicable the minimum vertical clearance shall be set at 1.0m clear headroom to assist in safe future inspections and maintenance activities.

## 12.2 Road Over Road Bridges and Footbridges

Horizontal clearances shall be in accordance with Figure 12.1 attached. The minimum clearance shown on Figure 12.1 is derived for highway and main roads. Clearances below those specified may be used if suitable traffic barriers are installed.

Traffic barriers may be flexible (e.g. wire rope safety barriers or w-beam) or rigid (eg: concrete barriers). Refer further DIS 3912/02-2 “Design of New Structures”, Section 2.3 and Road and Traffic Engineering Branch’s Document No. D11#38472, *MRWA Supplement to Austroads Guide to Road Design – Part 6*.

For any situation not covered in this section, reference may be made to AUSTROADS Guide to the Geometric Design of Major Urban Roads.

Vertical clearances shall be in accordance with Part 1, Clause 13.7 of the CODE, but shall be approved by the SD&SE.

## 12.3 Road Over Rail Bridges

Clearances shall be in accordance with the Public Transport Authority (PTA) clearance requirements. The clearance diagrams from PTA dated 28 Feb 2018 are shown at Figures 12.2 to 12.4. In all cases confirmation must be obtained from PTA.

On electrified routes, suitable protective screens must be installed on over-bridges to prevent people from touching the wires. Screens shall accommodate the minimum clearances as required by PTA. Screens must be provided on both sides of the bridge and must extend as per Public Transport Authority Specification “Protection Screens for Bridges over Electrified Railways”.

Because of the potentially catastrophic consequences of a train hitting a bridge pier, wherever possible railway lines should be crossed in a single span with solid abutments. Where a single span is not possible, piers shall be positioned and designed in accordance with the CODE and DIS 3912/02-2 “Design of New Structures”, Section 2.3.

## 12.4 Bridges Over Water

Navigation clearances are not usually a requirement for bridges in WA, but where they may be required, the relevant figures must be obtained from the Department for Planning and Infrastructure, Marine Information.

## 12.5 Clearances During Construction

During construction, it is often possible to reduce the above requirements. For road bridges, horizontal clearances may be reduced to the minimum required for safe working space with suitable protection works provided alternate routes are available for the transport of over-height vehicles. Vertical clearances may be reduced to 4.7 m or lower in specific circumstances. However, this may require special protective measures to be taken during construction and each case must be approved by SES. The preferred minimum clearance during construction is 4.90m.

For rail bridges, no reduction in horizontal clearance is permitted, however reduction in vertical clearance may be possible. Each case must be checked individually with PTA. In particular, construction over electrified lines will require special measures to be taken.

## 12.6 High Load Routes

There are a number of specific transport routes which are designated High Load Routes for the transport of large, indivisible loads. It is important that the vertical clearances on these routes are not encroached upon.

Details of existing and future preferred High Load Routes are shown in the attached Figures 12.5 and 12.6.

Designers shall ensure that all road bridges over these roads conform to these requirements. Clearances to footbridges should be an additional 300 mm greater than those shown.

The required vertical clearance shall be entered on the Design Criteria Sheet.

## 12.7 High/Wide Load Routes

High/Wide Load Routes incorporate the High Wide Load Corridor Project involving the development of suitable transport envelopes that will accommodate over-dimension loads up to 8m high, 8m wide and 24m long, and with a maximum 270 tonne net load along the designated routes that link key heavy industry centres in the Perth Metropolitan area. For other routes, clearance requirements need be obtained from Access Manager of Heavy Vehicle Services MRWA.

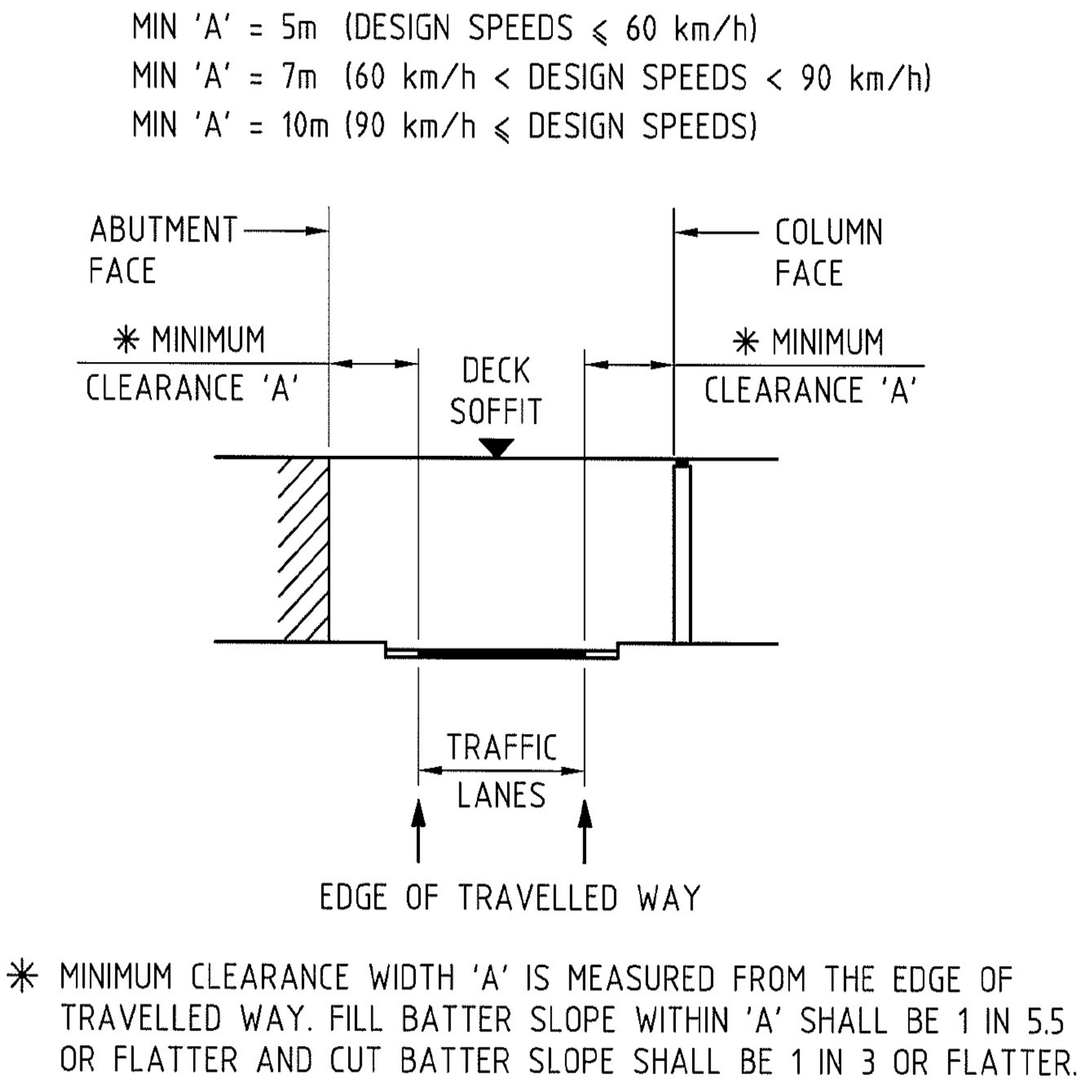
The designated routes and clearance requirements are located online at www.mainroads.wa.gov.au, Building Roads, Standards and Technical, Road and Traffic Engineering, Guide to Road Design, High Wide Loads.

The use of clearances for High/Wide Load vehicles for the design of new structures shall be confirmed with the Design Criteria Sheet and approved by the SES.

FIGURE 12.1

HORIZONTAL CLEARANCE TO SUBSTRUCTURE

COMPONENTS OF BRIDGES OVER ROADWAYS



FIGURES 12.2 TO 12.4
RAILWAY CLEARANCE REQUIREMENTS

# NARROW GAUGE LINES (See Sheet 1)

## 1. NOTES ON TRACK CENTRES

1.1 Desireable minimum track centres for tangent track.
  1.1.1 Adjacent tracks with marginal masts 4000mm.
  1.1.2 Central overhead traction masts 5200mm.

1.2 Minimum track centres in mm for curved track:

1.2.1 $4000 \text{ or } 5200 + \dfrac{72250}{R} + 3(Ea1-Ea2)$ Where Ea1 is greater than Ea2

1.2.2 $4000 \text{ or } 5200 + \dfrac{72250}{R}$ Where Ea2 is greater than Ea1

Ea1 = Cant in mm of Outer Track
Ea2 = Cant in mm of Inner Track
R = Radius in Metres

## 2. NOTES ON STRUCTURAL CLEARANCE OUTLINE

2.1 Minimum structural clearance of 2140mm to be increased by

$\dfrac{36125}{R} + 3Ea$ on the inside of a curve

Where R = Radius in metres  Ea = Cant in mm
(no increase to outside of curve)

2.2 All new overhead structures are to be constructed with a minimum overhead clearance in accordance with "The Design Supply Construction and Commissioning of 25 kvAC Traction Overhead Catenary Equipment" Doc. No. 8190-800-001.

2.3 Heights to be measured from high rail on curves.

2.4 Clearance outline to be increased to 4500mm from track centreline for fences and long structures without refuges. Faces of masts carrying catenary systems to be in accordance with "The Design Supply Construction and Commissioning of 25 kvAC Traction Overhead Catenary Equipment" Doc. No. 8190-800-001. For bridges and retaining walls within the railway reserve refer to Drg 00-C-04-0079

## 3. NOTES ON ROLLINGSTOCK & LOADING OUTLINES

3.1 All new passenger rollingstock shall be built to the Urban Passenger Railcar Static Outline.

3.2 The maximum longitudinal dimensions for rollingstock and railcar outlines are:
i) Length over body 24 metres
ii) Bogie Centres 17 metres
iii) Bogie Centre to body end 3.5 metres
The values in Notes 1.2 and 2.1 are based upon these dimensions.

## 4. NOTES ON PLATFORMS AND STATION STRUCTURES

4.1 Platform Clearances

4.1.1 Platforms are a permanent infringement of the base operating standard.

4.1.2 Platforms on curves must be approved in advance by the Manager Track and Structures or their approved delegate, as a minimum prior to Project Definition Plan stage.

4.1.3 On curves platform distance in mm from centre of track to be increased by:

4.1.3.1 Convex Platform Faces: $\dfrac{36125}{R} + Ea$

4.1.3.2 Concave Platform Faces: $\dfrac{36125}{R} - Ea$

R = Radius in metres
Ea = Cant in mm

Note that concessions to these dimensions may be required for conformance to disability standard at door steps.

4.2 Platform heights on curves to be measured perpendicular to the plane of the designed cant. i.e. top of rails.

4.3 For structures on platforms, see Drgs 00-C-04-1398 and 00-C-04-1399

# STANDARD AND DUAL GAUGE LINES (See Sheet 2)

## 5. NOTES ON TRACK CENTRES

5.1 Minimum track centres for tangent track
  5.1.1 Adjacent tracks 4000mm. NG to NG, SG to SG and NG to SG.
  5.1.2 Central overhead traction masts:
    Centres to provide for masts to be outside minimum structural clearance

5.2 Minimum track centres in mm for curved track
  5.2.1 Passenger Rollingstock and Loading Outline

5.2.1.1 Tangent Centres $+ \dfrac{83600}{R} + 3(Ea1-Ea2)$ Where Ea1 is greater than Ea2

5.2.1.2 Tangent Centres $+ \dfrac{83600}{R}$ Where Ea2 is greater than Ea1

Ea1 = Cant in mm of Outer Track  Ea2 = Cant in mm of Inner Track
R = Radius in Metres

  5.2.2 Freight Rollingstock and Loading Outline

5.2.2.1 Tangent Centres $+ \dfrac{83600}{R} + 4.5\,(Ea1-Ea2)$ Where Ea1 is greater than Ea2

5.2.2.2 Tangent Centres $+ \dfrac{83600}{R}$ Where Ea2 is greater than Ea1

## 6. NOTES ON STRUCTURAL CLEARANCE OUTLINE

6.1 Minimum structural clearance of 2140mm, 2325mm or 2500mm to be increased by

$\dfrac{41800}{R} + 3Ea$ or $\dfrac{41800}{R} + 4.5\,Ea$ (Nth Fremantle-Robb Jetty)
on the inside of a curve

Where R = Radius in metres
Ea = Cant in mm
(no increase to outside of curve)

6.2 All new overhead structures are to be constructed with a minimum overhead clearance in accordance with "The Design Supply Construction and Commissioning of 25 kvAC Traction Overhead Catenary Equipment" Doc. No. 8190-800-001.

6.3 Heights to be measured from high rail on curves.

6.4 Clearance outline to be increased to 4500mm from track centreline for fences and long structures without refuges. Faces of masts carrying catenary systems to be in accordance with "The Design Supply Construction and Commissioning of 25 kvAC Traction Overhead Catenary Equipment" Doc. No. 8190-800-001. For bridges and retaining walls within the railway reserve refer to Drg 00-C-04-0079

6.5 On the common rail side the co-incident NG clearance outline governs.

## 7. NOTES ON ROLLINGSTOCK & LOADING OUTLINES

7.1 Current minimum wire height of 6200 on the North Fremantle to Fremantle Section precludes operation of double stack container vehicles. For vehicle outline refer to AS7507 RISSB Reference vehicle 33.

7.2 The maximum longitudinal dimensions for rollingstock and loading outlines are:
i) Length over body 25.9 metres
ii) Bogie Centres 18.3 metres
iii) Bogie Centre to body end 3.8 metres
The values in Notes 5.2 and 6.1 are based upon these dimensions.

## 8. NOTES ON PLATFORMS AND STATION STRUCTURES

8.1 Platform Clearances

8.1.1 Platforms are a permanent infringement of the base operating standard.

8.1.2 The platform clearance and platform height shown are for standard gauge track only. For dual gauge track, narrow gauge clearances govern. See Sheet 1 for narrow gauge clearances.

8.1.3 On curves platform distance in mm from centre of track to be increased by:

8.1.3.1 Convex Platform Faces: $\dfrac{41800}{R} + Ea$

8.1.3.2 Concave Platform Faces: $\dfrac{41800}{R} - Ea$

R = Radius in metres
Ea = Cant in mm

Note that concessions to these dimensions may be required for conformance to disability standard at door steps.

8.2 Platform heights on curves to be measured perpendicular to the plane of the designed cant. i.e. top of rails.

8.3 For structures on platforms, see Drgs 00-C-04-1398 and 00-C-04-1399

# GENERAL NOTES (Sheets 1 and 2)

## 9. GENERAL NOTES

9.1 For special circumstances, concessional structural clearances may be considered by the Manager Track and Structures (or their approved delegate) with a supporting safety argument and associated risk assesssment. These shall comply with the general design procedures of the NG & SG Codes of Practice. Approved concessions shall be entered into the clearance register. Use standard form 8830-400-007 - Request for Concessional Structural Clearance

9.2 Third Party access. Procedures must be in accordance with the Railways (Access) Act 1998 and Railways (Access) Code 2000.

9.3 For Clearance and Rollingstock Outlines on Narrow Gauge Lines, refer 00-C-04-0076

9.4 For Clearance and Rollingstock Outlines on Standard and Dual Lines, refer 00-C-04-0077

| REV | DATE | AMENDMENT | DSN | DRN | CHKD | APP |
|---|---|---|---|---|---|---|
| 1 | 28.02.2018 | ISSUED AS STANDARD | ZS | ZS | MK | RD |
| 0 | 19.05.06 | ISSUED FOR CONSTRUCTION | WPL | AWP | WPL | PM |

ORIG SIZE A1 — 0 10 20 30 40 50 100mm AT ORIGINAL PLOT SIZE



REFERENCES

SCALE : NTS (@ A1)
DATUM :
HORIZONTAL:
VERTICAL:
PTA ACCEPTANCE STAMP A3335619

DESIGNED
DRAWN
CHECKED
APPROVED FOR ISSUE
MANAGER TRACK & STRUCTURES
Network & Infrastructure
DATE

Government of Western Australia
Public Transport Authority

CLEARANCES
STANDARDS FOR URBAN NETWORK
STRUCTURAL CLEARANCE AND ROLLINGSTOCK OUTLINES
NOTES RELATING TO CLEARANCES

PTA Drawing No: 00-C-04-0078 REV : 1

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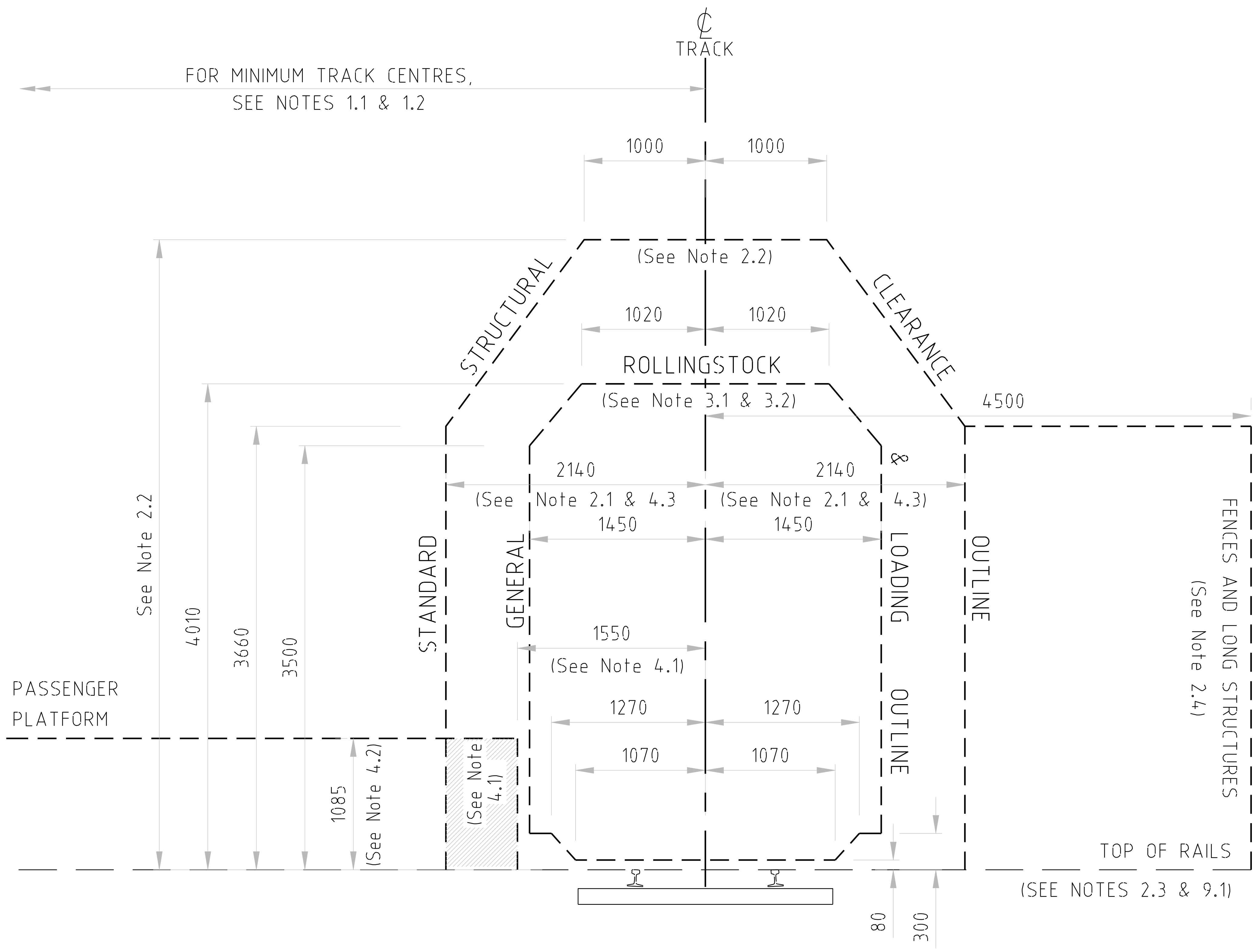
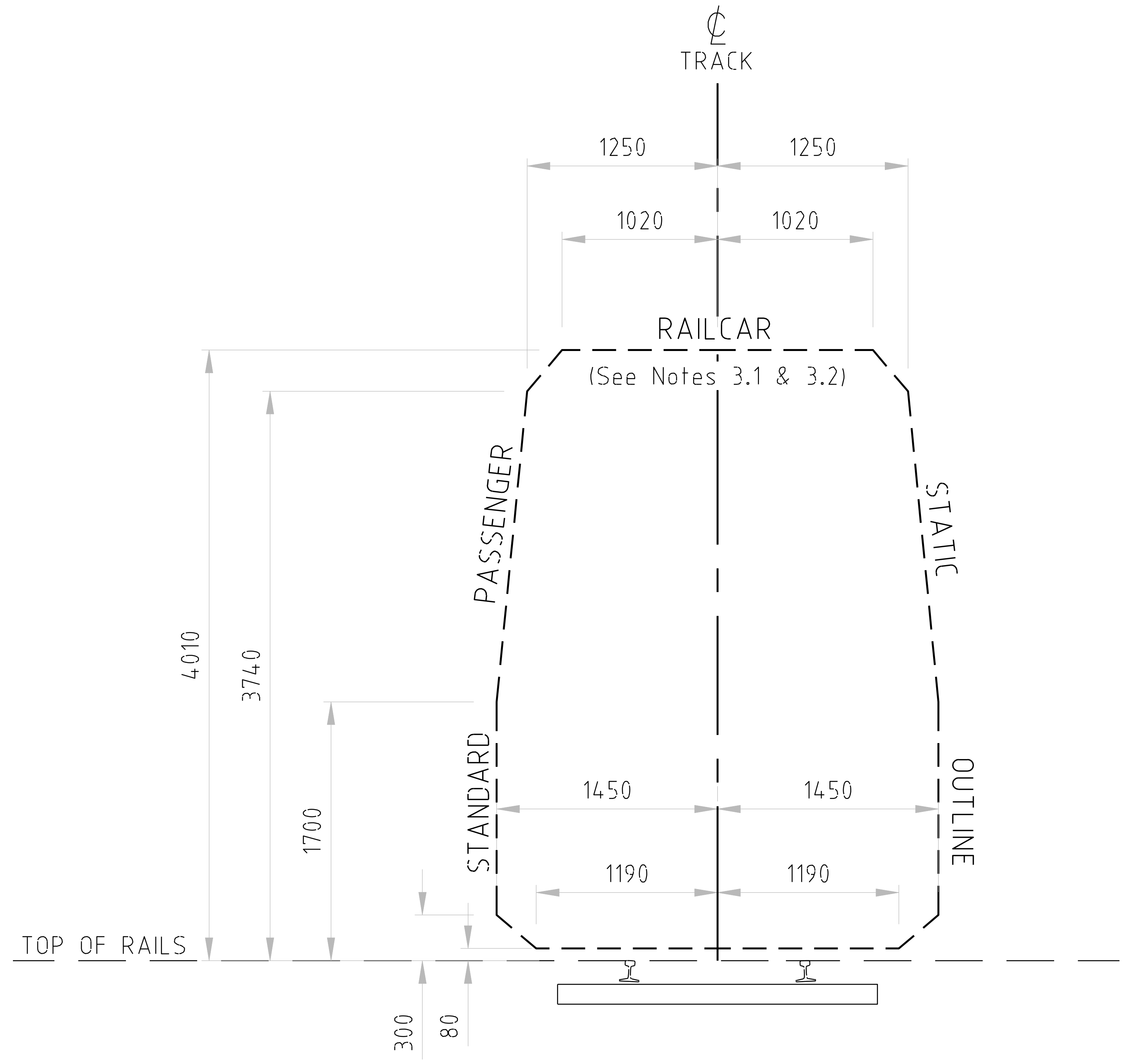



STANDARD OUTLINES FOR STRUCTURAL CLEARANCE, ROLLINGSTOCK AND LOADING




STATIC OUTLINE FOR STANDARD PASSENGER RAILCARS

## NOTES

1. All dimensions shown on this drawing are in mm.
2. For referenced notes refer drawing 00-C-04-0078
3. For Clearance and Rollingstock Outlines on Standard and Dual Gauge Lines refer drawing 00-C-04-0077
4. Refer to AS 7507, RISSB reference vehicle 13, 14 and 31 for Narrow Gauge vehicles

| REV | DATE | AMENDMENT | DSN | DRN | CHKD | APP |
|---|---|---|---|---|---|---|
| 1 | 28.02.2018 | ISSUED AS STANDARD | ZS | ZS | MK | RD |
| 0 | 19.05.06 | ISSUED FOR CONSTRUCTION | WPL | AWP | WPL | PM |

ORIG SIZE A1 — 0 10 20 30 40 50 100mm — AT ORIGINAL PLOT SIZE



REFERENCES

SCALE : 1:25 (@ A1)
DATUM :
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VERTICAL:
PTA ACCEPTANCE STAMP
A3335619

DESIGNED
DRAWN
CHECKED
APPROVED FOR ISSUE
MANAGER TRACK & STRUCTURES
Network & Infrastructure
DATE

Government of Western Australia
Public Transport Authority

CLEARANCES
STANDARDS FOR URBAN NETWORK
STRUCTURAL CLEARANCE AND ROLLINGSTOCK OUTLINES
NARROW GAUGE LINES

PTA Drawing No: 00-C-04-0076 REV : 1

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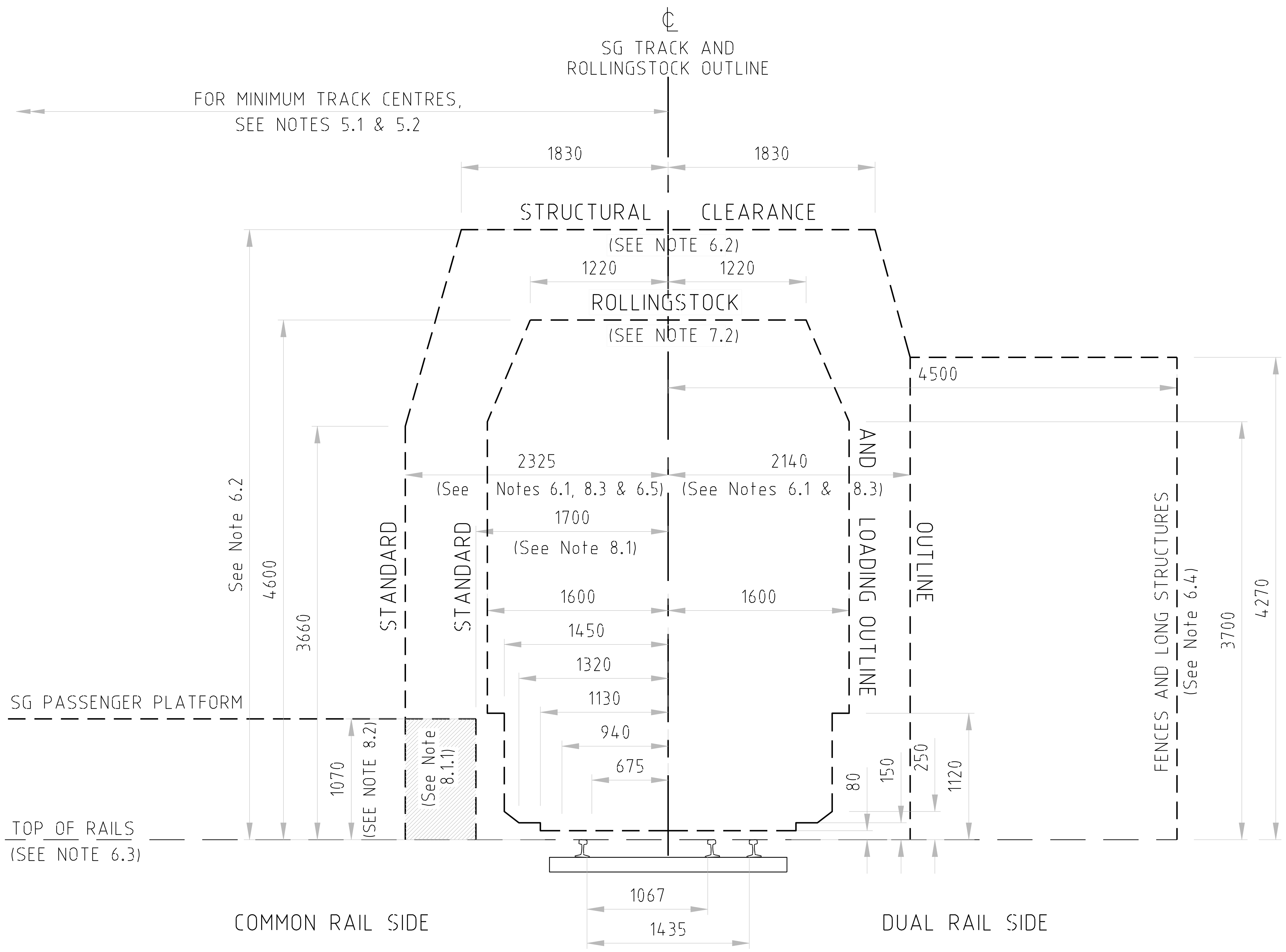


# EAST PERTH - MIDLAND RAILWAY

## DUAL GAUGE STRUCTURAL & STANDARD GAUGE ROLLINGSTOCK OUTLINES

### NOTES

1. All dimensions shown on this drawing are in mm.
2. For referenced notes refer drawing 00-C-04-0078
3. For Clearance and Rollingstock Outlines on Narrow Gauge Lines refer drawing 00-C-04-0076
4. North Fremantle to Robb Jetty Railway - see note 7.1
5. Refer to AS 7507, RISSB reference vehicle 15 for Dual Gauge vehicles

| REV | DATE | AMENDMENT | DSN | DRN | CHKD | APP |
|---|---|---|---|---|---|---|
| 1 | 28.02.2018 | ISSUED AS STANDARD | ZS | ZS | MK | RD |
| 0 | 19.05.06 | ISSUED FOR CONSTRUCTION | WPL | AWP | WPL | PM |

ORIG SIZE A1 — 0 10 20 30 40 50 100mm AT ORIGINAL PLOT SIZE



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REFERENCES

SCALE : 1:25 (@ A1)

DATUM :

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DESIGNED

DRAWN

CHECKED

APPROVED FOR ISSUE

MANAGER TRACK & STRUCTURES Network & Infrastructure

DATE

Government of Western Australia Public Transport Authority

CLEARANCES

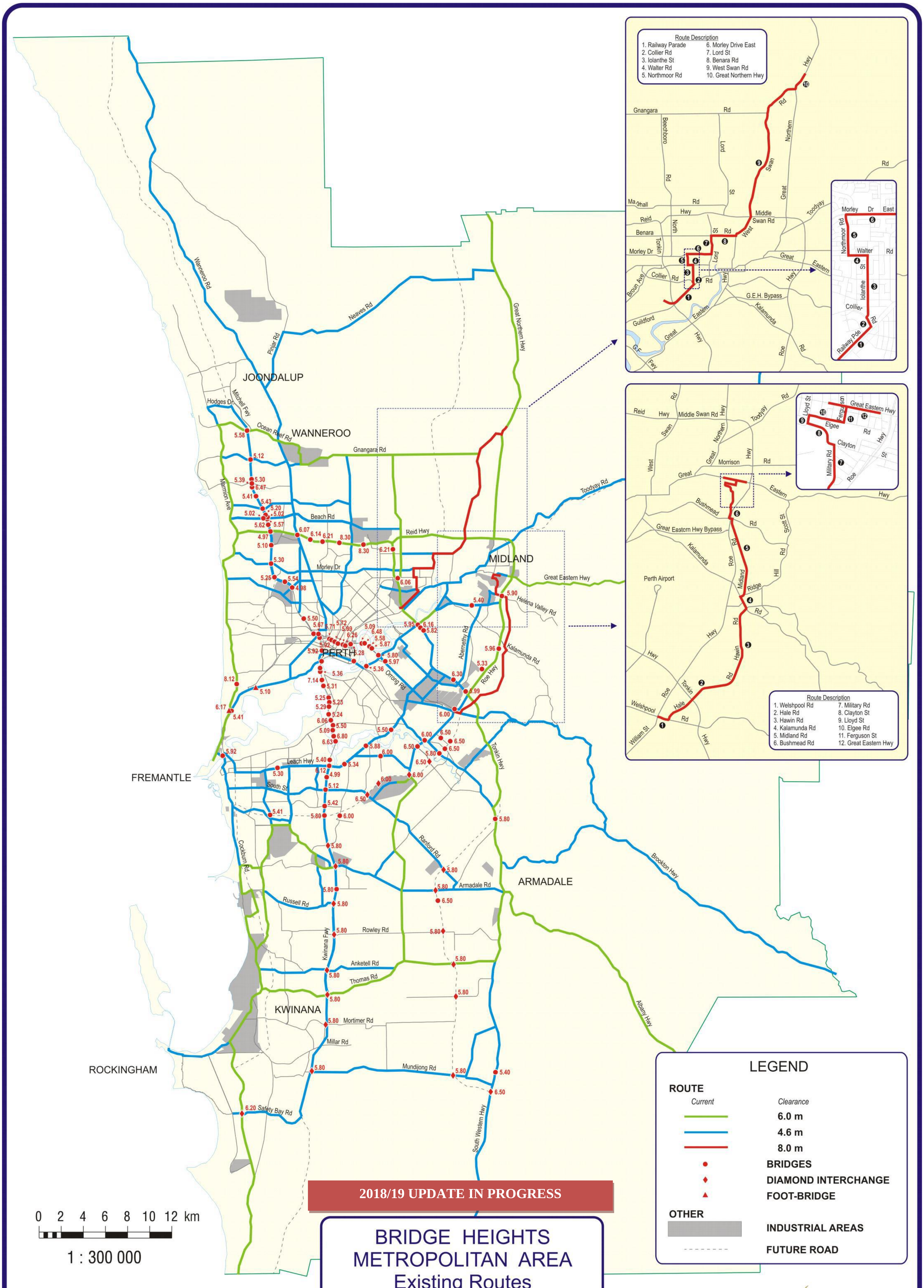
STANDARDS FOR URBAN NETWORK

STRUCTURAL CLEARANCE AND ROLLINGSTOCK OUTLINES

STANDARD AND DUAL GAUGE LINES

PTA Drawing No: 00-C-04-0077 REV : 1

FIGURE 12.5
METROPOLITAN BRIDGE HEIGHTS - EXISTING & PROPOSED




# BRIDGE HEIGHTS METROPOLITAN AREA

## Existing Routes

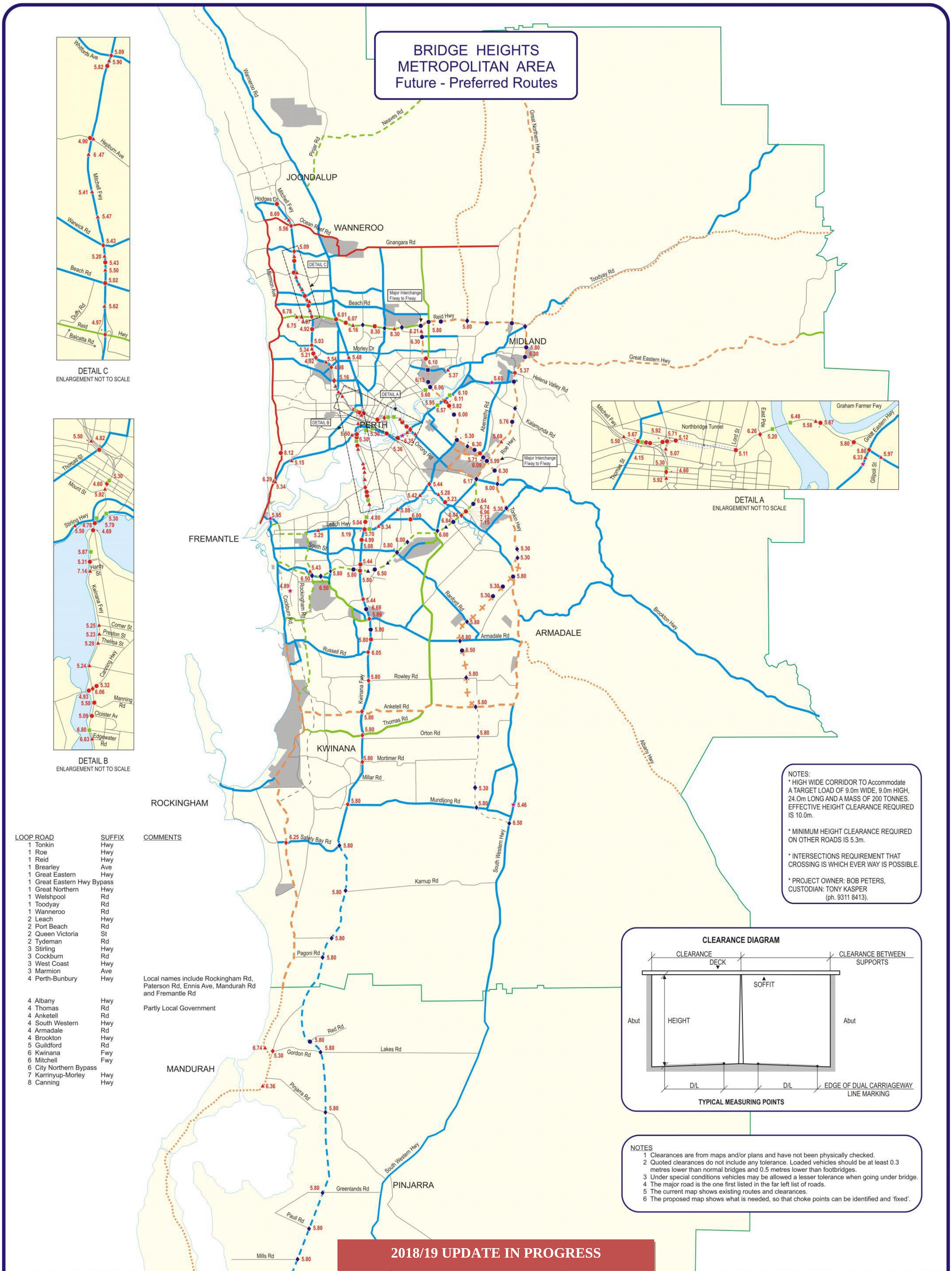
Date: 30/4/2001

ASSET AND NETWORK INFORMATION

Planning & Practice Directorate
Project: ANI/RIS/Projects/J2501/dwg00930382-1.cdr
Data Source: Road Transport Section
Data Currency: April 2001
Dwg No: 0093-0382
Date of Print: February, 2002

MAIN ROADS Western Australia

# BRIDGE HEIGHTS METROPOLITAN AREA
## Future - Preferred Routes




| LOOP ROAD | | SUFFIX | COMMENTS |
|---|---|---|---|
| 1 | Tonkin | Hwy | |
| 1 | Roe | Hwy | |
| 1 | Reid | Hwy | |
| 1 | Brearley | Ave | |
| 1 | Great Eastern | Hwy | |
| 1 | Great Eastern Hwy Bypass | | |
| 1 | Great Northern | Hwy | |
| 1 | Welshpool | Rd | |
| 1 | Toodyay | Rd | |
| 1 | Wanneroo | Rd | |
| 2 | Leach | Hwy | |
| 2 | Port Beach | Rd | |
| 2 | Queen Victoria | St | |
| 2 | Tydeman | Rd | |
| 3 | Stirling | Hwy | |
| 3 | Cockburn | Rd | |
| 3 | West Coast | Hwy | |
| 3 | Marmion | Ave | |
| 4 | Perth-Bunbury | Hwy | Local names include Rockingham Rd, Paterson Rd, Ennis Ave, Mandurah Rd and Fremantle Rd |
| 4 | Albany | Hwy | |
| 4 | Thomas | Rd | Partly Local Government |
| 4 | Anketell | Rd | |
| 4 | South Western | Hwy | |
| 4 | Armadale | Rd | |
| 4 | Brookton | Hwy | |
| 5 | Guildford | Rd | |
| 6 | Kwinana | Fwy | |
| 6 | Mitchell | Fwy | |
| 6 | City Northern Bypass | | |
| 7 | Karrinyup-Morley | Hwy | |
| 8 | Canning | Hwy | |

NOTES:
* HIGH WIDE CORRIDOR TO Accommodate A TARGET LOAD OF 9.0m WIDE, 9.0m HIGH, 24.0m LONG AND A MASS OF 200 TONNES. EFFECTIVE HEIGHT CLEARANCE REQUIRED IS 10.0m.

* MINIMUM HEIGHT CLEARANCE REQUIRED ON OTHER ROADS IS 5.3m.

* INTERSECTIONS REQUIREMENT THAT CROSSING IS WHICH EVER WAY IS POSSIBLE.

* PROJECT OWNER: BOB PETERS, CUSTODIAN: TONY KASPER (ph. 9311 8413).

**CLEARANCE DIAGRAM**

CLEARANCE
DECK
CLEARANCE BETWEEN SUPPORTS
SOFFIT
Abut
HEIGHT
Abut
D/L
D/L
EDGE OF DUAL CARRIAGEWAY LINE MARKING

TYPICAL MEASURING POINTS

NOTES
1. Clearances are from maps and/or plans and have not been physically checked.
2. Quoted clearances do not include any tolerance. Loaded vehicles should be at least 0.3 metres lower than normal bridges and 0.5 metres lower than footbridges.
3. Under special conditions vehicles may be allowed a lesser tolerance when going under bridge.
4. The major road is the one first listed in the far left list of roads.
5. The current map shows existing routes and clearances.
6. The proposed map shows what is needed, so that choke points can be identified and 'fixed'.

**2018/19 UPDATE IN PROGRESS**

## LEGEND

0 2 4 6 8 10 12 km
SCALE

DRAFT:
Internal Use Only
Date: 8/8/2003

ROUTE
Existing | Proposed | Clearance
10.0 m
10.0 m (Alternative)
10.0 m (Future considerations)
8.0 m
6.0 m
4.6 m

BRIDGES
Existing | Proposed
Restricted movement
Unrestricted movement in the direction of diamond orientation (See Direction Diagram)
Footbridge

BRIDGES
Existing
Sign gantry
Railway bridge over road

OTHER
Industrial area

'B'
'A'
Bridge is on route 'A' over route 'B', unrestricted movement in the direction of the diamond orientation.
DIRECTION DIAGRAM

MAIN ROADS Western Australia
ASSET AND NETWORK INFORMATION

# SECTION 13 – BRIDGE WATERWAYS INVESTIGATION AND FLOOD ESTIMATION

This information is Part 13 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Senior Waterways Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 15/08/18

**Document No: 3912/02-13**

**SECTION 13**

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 09/01/06 | Complete review |
| All | 2 | 15/08/18 | Complete review for introduction of AS5100-2017 |
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Custodian Endorsement

E. CHEUNG
A/Senior Waterways Engineer
Date: 15/08/18

# 13 BRIDGE WATERWAYS INVESTIGATION AND FLOOD ESTIMATION

## 13.1 Bridge Waterways Design Computer Programs

### 13.1.1 Introduction

This section contains a brief outline of some of the suitable and common waterways programs used for hydrological and hydraulic analyses to design bridges for Main Roads. A detailed manual is available for most of the programs giving full information on theoretical background, inputs and outputs.

The programs mentioned in this section are not exclusive. There are other waterways programs that are also suitable and acceptable for hydrological and hydraulic analyses to design bridges for Main Roads.

It is recommended that this section be read in conjunction with Austroads Guide to Bridge Technology Part 8 – Hydraulic Design of Waterway Structures – Section 3.6: Computer Modelling. Other sections in the Austroads Guide which are recommended for reading include Section 2.2: Estimation of Design Floods, Section 4.3: Analysis Methods, Section 4.4: 1D Bridge Hydraulic Analysis and Section 4.5: 2D Bridge Hydraulic Analysis.

### 13.1.2 HEC-RAS (River Analysis System)

This program can calculate water surface profiles for steady gradually varied flow in channels in a one dimensional (1D) model. Profiles may be calculated for both sub critical and super critical flow. The program can model obstructions such as bridges, culverts, weirs and levees. The computational procedure is based on the solution of the one dimensional energy equation with energy loss due to friction evaluated with Manning's Equation.

This program can also calculate water surface and velocity profiles for unsteady flow in channels and wider floodplains in a 1D, two dimensional (2D) and combined 1D/2D model. Full Shallow Water or Diffusion Wave equations may be selected for the computational procedure, where full Shallow Water is applicable to a wider range of problems and Diffusion Wave allows the program to run faster and have greater stability properties. Flows can be generated in the 2D model through hydrograph and direct rainfall inputs.

### 13.1.3 HEC-HMS (Hydrologic Modeling System)

This program simulates the hydrologic processes of watershed systems utilising traditional hydrologic analysis procedures including event infiltration, unit hydrographs, and hydrologic routing to generate hydrographs which can be input into hydraulic models. Advanced capabilities are available for gridded runoff simulation using the linear quasi-distributed runoff transform method (ModClark). Supplemental analysis tools are also available for model optimisation, forecasting streamflow, depth-area reduction, assessing model uncertainty, erosion and sediment transport, and water quality.

### 13.1.4 HEC-SSP (Statistical Software Package)

This program performs statistical analyses of hydrologic data and can calculate the expected probability curve and confidence limits of flood flow. The program can perform flood frequency analysis based on Bulletin 17B (Interagency Advisory Committee on Water Data, 1982), Bulletin 17C (England, et al., 2015) and also a generalised frequency analysis on not only flow data but other hydrologic data as well. Both Bulletin's recommend log-Pearson Type III distribution however Bulletin 17C advances in several areas including adopting the Expected Moments Algorithm and Multiple Grubbs Beck Test to address low outliers, and correcting confidence intervals for the flood frequency curve. It is recommended that the practitioner check that the results output from this program are in accordance with Australian guidelines.

### 13.1.5 RORB (Runoff Routing 'B')

This program is based on runoff routing concepts and is used for hydrological analysis of catchment areas to produce design hydrographs. The model is aerially distributed, nonlinear, and applicable to both urban and rural catchments. The program has the capability of simulating storage and allows 'DESIGN' runs using design rainfall and temporal patterns to estimate design flows, or 'FIT' runs where actual rainfall events may be fitted to known flows in order to estimate catchment parameters. The program also has capability to vary parameters over sub-catchments and carry out batch runs and Monte Carlo simulations.

### 13.1.6 TUFLOW

This program can calculate water surface and velocity profiles in channels and wider floodplains in a 1D, 2D and combined 1D/2D model. It is ideally suited to modelling flooding of rivers and creeks with complex flow patterns, overland and piped flows through urban areas and coastal scenarios. The program can model obstructions such as bridges and culverts in 1D models, and also in 2D models provided that the flow width of the structure is of similar or larger size than the 2D cell size. The 1D Solver utilises the Saint-Venant equations and the 2D implicit solver is based on Stelling (1984) and Syme (1991) and solves the full two-dimensional, depth averaged, momentum and continuity equations for free-surface flow. The models can be run using several engines including Classic, HPC (Heavily Parallelised Compute) and FV (Finite Volume) to optimise run times and obtain results as appropriate.

### 13.1.7 TUFLOW FLIKE

This program calculates the probability of flood events based on historical records to calculate the expected probability curve and confidence limits of flood flow. The primary purpose of the program is to carry out flood frequency analysis however it can be applied to extreme value analysis. A range of probability models distributions can be applied to the analysis. The program uses the Bayesian inference methodology which allows the ability to use historic data outside of the gauged record, incorporate the multiple Grubbs-Beck test for low outliers and incorporate regional information as prior knowledge.

### 13.1.8 MIKEFLOOD

The MIKEFLOOD package contains several 1D and 2D simulation engines which can model rivers, open channels, sewer and drainage networks, and overland flow. These engines can be coupled with one another to create an integrated model to calculate water surface and velocity profiles in channels and wider floodplains. The program can model obstructions such as bridges, culverts and weirs. The package can also be used to design and assess coastal structures, offshore structures, dams and environmental flood impacts. The 2D component of MIKEFLOOD, MIKE 21 HD, is a general numerical modelling system that utilises the conservation of mass and momentum integrated over the vertical equations to describe flow and water level variations. Run times can be optimised by enabling parallel 2D engines.

### 13.1.9 XP-STORM

This program can simulate stormwater and river flows using models comprising of 1D channels and pipes coupled to a 2D surface grid. All hydrologic processes including snowmelt, evaporation, infiltration, surface ponding and ground-surface water exchanges can be included in the model. Rainfall can be selected through design storms or actual recorded rainfall events, and the program provides numerous methods for computing runoff from the rainfall. This program utilises the Saint-Venant equation as the hydraulic computational procedure in 1D models to calculate backwater effects, flow reversal and surcharging. The program can model obstructions and control structures such as bridges, culverts, and detention basins in the 1D model. The 2D modelling package incorporates the TUFLOW engine into the XP graphical interface.

### 13.1.10 XP-RAFTS

This program may be used to simulate runoff hydrographs at defined points throughout a watershed for a set of catchment conditions and rainfall events and is suitable for application on both rural and urbanised catchments of all sizes. The watershed can be divided into a number of sub-catchments and storage both small and large in volume may be assigned to nodes. Rainfall events can be generated from Intensity-Frequency-Duration data together with storm temporal patterns or standard pre-set storm data. The program uses the Laurenson non-linear runoff routing procedure to develop hydrographs and a number of loss models can be selected to calculate rainfall excess.

### 13.1.11 AFFLUX

This program carries out surface water analysis in channels based on Manning's Equation and also computes backwater as a result of obstructions. The program models the natural stream properties and superimposes a bridge template over this natural stream. The bridge template includes information such as deck level, abutment chainages, abutment type, number of piers and skew. Scour and floodways may also be added to the model. Outputs from this program consists of discharge, velocities and backwater height for each specified stage height. It is recommended that this program only be considered for simple scenarios of bridges operating with or without floodways.

### 13.1.12 CRC-FORGE FOR WESTERN AUSTRALIA

This tool is a regional frequency analysis method used for estimating large to rare rainfalls in Western Australia. This method is based on the concept that additional information can be gained by pooling standardised data from a number of rainfall sites at a regional scale and standardised. Growth curves of design rainfalls are generated by plotting at-site data and pooling additional data from rainfall sites within areas of increasing size to estimate rainfalls with decreasing Annual Exceedance Probabilities (AEP). A seasonal approach to extreme flood estimation has specifically been adopted for Western Australia due to rainfall characteristics evident in this state.

### 13.1.13 RFFE (Regional Flood Frequency Estimation Model)

This model calculates discharge and provides confidence limits based on catchment location and characteristics using data from relevant gauged catchments. The application of this model is most relevant in ungauged catchment scenarios. This model provides an approach which is consistent nationally and smooths discrepancies at state boundaries which previously existed. The Region of Influence approach is adopted to iteratively determine the lowest prediction error and maximise the regression model predictive skill. The reliability of results is highly dependent on data availability and quality. This model is recommended in the Australian Rainfall and Runoff 2016 guidelines for ungauged catchments and the practitioner must recognise and understand the limitations of this model prior to using it.

There is no specific program to calculate discharge using the regional methods in the Australian Rainfall and Runoff 1987 guidelines, however it is noted that Main Roads currently still recommends practitioners carry out these calculations in conjunction with the RFFE model.

# SECTION 14 – STRUCTURAL DESIGN AND ANALYSIS SOFTWARE

This information is Part 14 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 16/04/18

**Document No: 3912/02-14**

**SECTION 14**

**CONTENTS**



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 13/12/05 | Complete review for introduction of AS 5100-2004 |
| All | 2 | 13/04/18 | Complete review for introduction of AS 5100-2017 |
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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 13/04/18

# 14 BRIDGE STRUCTURAL DESIGN COMPUTER PROGRAMS

## 14.1 INTRODUCTION

This Section contains a brief outline of structural design and analysis software programs normally used within Main Roads.

Structures Engineering prefers to receive computer models prepared using the below programs so that the models can be used internally if required.

For full information on input, output and theoretical background, refer to program user guides and manuals.

## 14.2 ACES

This is a general structural analysis software package performing linear elastic static analysis. Applications include 2 and 3 dimensional frames, grillage and finite element analysis of plane, plate and shell structures.

ACES-BEAM provides a continuous bridge beam module for linear elastic analysis with any span configuration.

ACES Incremental Launching module calculates and displays effects of an incrementally launched girder, using an iterative linear-elastic analysis.

ACES Section Properties module allows properties to be calculated of an arbitrary shaped section, including hollow sections, and sections with a number of voids.

The basic input is composed of the following:

- Geometry of the structure - defined by a series of nodes, connected by members or elements.
- Physical properties of all members and/or elements.
- Location and types of support (rigid or elastic).
- Load data, including self-weight, temperature, differential settlement and traffic load. The program has several in-built AS 5100, Bridge Design (CODE) traffic vehicles and facility to set-up standard vehicles for load application.

Output consists of all nodal, member and/or element displacements and forces (moments, shear and reactions). The program has graphical facilities for mesh generation, vehicle loadings, to validate input data and to plot output information.

## 14.3 COLDES

This program performs section analysis and assists with the design of 'short' reinforced concrete columns. It calculates the ultimate strength, ultimate balanced strength and minimum reinforcement to take the applied axial and/or bending moment, and plots the axial-bending moment interaction diagram. Torsion and shear are not considered.

It also provides a screen plot of the section to enable checking of input data and axial-bending moment interaction diagram to enable selection of a suitable column.

## 14.4 CONKS

This program carries out a cracked section analysis of reinforced and prestressed concrete under serviceability and ultimate moments.

Output gives the ultimate moment capacity of the section plus concrete and steel stresses under serviceability moments.

## 14.5 CSI BRIDGE

This is an advanced software package produced by Computers & Structures Inc. for use in more complex projects. Useful for analysis, design, load rating and reporting.

## 14.6 GRLWEAP

A program for analysing dynamic pile driving response. Inputs consist of soil profile (shear modulus), pile details (weight, length material etc), hammer (rated energy, weight of ram, type etc). Outputs are pile driving response, set, pile stresses etc.

The program is useful in selecting the appropriate pile driving hammer, helmet, packing material etc to achieve the nominated pile capacity, and to design the pile for driving conditions.

## 14.7 HLR

HLR is a computer program to enable indicative assessment of load-carrying capacity of bridge structures and to monitor passage of heavy vehicles on a defined road network.

## 14.8 LIMSTEEL

Interactive design of steel members to AS 4100.

## 14.9 LOADIST

This program interactively performs Guyon-Massonet analysis of an orthotropic simply supported bridge superstructure. The analysis is linear elastic. It is used for determining the distribution of longitudinal moments due to traffic load in a multi-beam bridge structures. MRWA only recommends its use for distribution factor comparative analysis.

## 14.10 MULTBEAM

Transverse distribution of longitudinal movements in linked plank bridges. Cannot input specific vehicles but good as a check for distribution factors from ACES.

## 14.11 PARTIAL

This program analyses partially prestressed continuous beams. It will allow for non-linear effects due to differential temperature gradient and shrinkage with cracking, plasticity and creep of both concrete and steel. Because of the general nature of the analysis it will also handle reinforced and fully prestressed beams and columns and analyse effects due to constructing staging.

Input consists of:

- Definition of structure - the beam must be subdivided into a number of sections longitudinally and the cross-section modelled as a number of layers with specified width and thickness.
- Definition of steel.
- Applied external loads, prestress forces, sequence of construction/staging, etc.

Output gives a prestress friction analysis, the deflected shape of the beam, moments and shears. The stresses and strains in each layer of steel and concrete can also be obtained at selected sections.

The program uses an iterative method of analysis aimed at achieving equilibrium throughout the structure under the given loading conditions and material's stress-strain relationship.

## 14.12 PCBEAMAN

This program performs linear elastic analysis of a continuous beam with any span configuration. It allows for varying moment of inertia, construction staging of a multi-stage bridge and composite section (beam and slab acting together).

Input is composed of the span configuration, section properties (moment of inertia, area, etc), details of construction staging if necessary and the applied external loads. The program has several in-built live loads including NAASRA traffic vehicles, and facility to enable setting-up of a user defined vehicle for load application. The program also has an in-built differential settlement facility.

Output gives the bending moments, shear forces, deflections and normal and shear stresses at the specified or result points and reactions of supports. The result can be presented in table format and/or graphically. The graphical viewing enables determination of position of a live load for maximum moment, shear or deflection and enables assessment of the variation of bending moments, shears and deflections caused by a moving live load.

## 14.13 PIGLET

This program analyses the load/deformation response of a pile group under general loading conditions.

Input consists of data on the pile group, soil parameters and loading. The pile cap is assumed to be rigid, with the piles pinned or fixed to the pile cap. The group may contain up to 85 piles, which can be vertical or raked in any direction. Coordinates and structural data are required. The only soil parameters necessary are Poisson's ratio and a shear modulus profile with depth. Only linear profiles are permissible. Loading may be general 3-D loads to the pile cap or individual piles or imposed deformations to the pile cap or individual piles.

Both a full and a summarised output are available. The "slimline" output gives the forces and moments in each pile and deformations of the pile cap. Full output gives the response of the group to unit deformations of the pile cap and overall stiffness and flexibility matrices for the group.

Analysis is based on a number of approximate, compact solutions for the response of single piles, with due allowance for interaction effects. The soil is assumed to behave as a linear elastic medium.

## 14.14 RETWALL

Interactive design of concrete retaining walls.

## 14.15 SPACEGASS

General purpose frame analysis program.

## 14.16 STLBEAM

STLBEAM performs static elastic analysis of a prismatic continuous bridge using a Fourier Transform of loading and support conditions. The analysis is based on harmonic folded plate theory. Applications include analysis of steel, concrete or composite bridge superstructures, particularly box girders.

The structure must be square at the abutments and prismatic. The structure is modelled as a collection of folded plates which may be isotropic or orthotropic. Longitudinal beam members can also be included in the assembly. Internal supports can be rigid or elastic, but only knife-edge supports are allowed at the abutment.

Advantages of STLBEAM over finite element programs are in the speed of modelling (using the graphic user interface and vehicle load generation) and the accuracy and speed of the analysis for transverse effects of vehicle loadings.

Results can be selected interactively for viewing and plotting. The output includes moments, forces plate stresses, displacements, etc.

## 14.17 STRUCTURAL BRIDGE DESIGN

This is an advanced software package produced by Autodesk for use in more complex projects. Useful for analysis, design, load rating and reporting.

## 14.18 TIMBAR

An interactive program using an ACES analysis engine for the rating and refurbishment design of timber bridges.

## SECTION 15 – INVENTORY INFORMATION

This information is Part 15 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Bridge Condition Manager is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 16/04/18

**Document No: 3912/02-15**

**SECTION 15**

## CONTENTS



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| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| 1 to 10 | 1 | 25/10/04 | Section 15 revised including the incorporation of the revised Section 16. |
| 3, 5, 6, 11 & 12 | 2 | 28/10/05 | Attachment 5 - General Bridge Inventory Information Amended.<br>Attachment 4 - Tunnel Inventory Information form added. |
| All | 3 | 14/12/05 | Complete review for introduction of AS 5100 |
| 5-8, 11 & 12 | 4 | 22/06/06 | Amended General Bridge, Culvert and Tunnel Inventory forms |
| 5-12 | 5 | 13/01/12 | Amended General Bridge, Culvert and Tunnel Inventory and Construction Information Forms |
| All | 6 | 12/04/18 | Complete Review |
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Custodian Endorsement

G. Johnston
Bridge Condition Manager
Date: 12/04/18

# 15 INVENTORY INFORMATION

## 15.1 Purpose

This Section defines the extent of information to be supplied for the Main Roads Bridge Inventory System to allow for efficient management of the State's infrastructure.

## 15.2 Scope

This Section shall be followed as part of the completion of the design of any bridge, tunnel, or culvert replacing an existing bridge, within or over a public road within WA.

## 15.3 References

This Section is provided for reference by the following Procedures of the Structures Engineering Management System Document 3912/01:

- Procedure for the Management of Bridge Data & Information 3912/01/04
- Procedure for the Design of Structures 3912/01/05
- Procedure for Design Review 3912/01/11
- Procedure for Updating Bridge Inventory and Construction Information 3912/01/12

## 15.4 Definitions

IRIS - Integrated Road Information System, which contains a database of inventory for each bridge from information supplied in accordance with this Section.

Designer - The person with sufficient knowledge of the design of the bridge to be able to provide the required information accurately.

Project Manager - Generic title referring to the person responsible for the delivery of the structural project.

Ownership – The Owner in IRIS is defined as the party responsible for maintenance, not the party that owns the asset.

Tunnel - A structure is defined as a Tunnel where its principal function provides access for road or rail and is generally buried within and surrounded by soil. Pedestrian and fauna under/overpasses are not recorded as tunnels in IRIS.

Rail Carriage – A superstructure formed using the undercarriage of a rail car.

Other Composite Steel Beams & Concrete Deck – This is for structures that are not composite I-beams, box girders, cable stayed or rail carriage. The only occurrence at time of writing is Third Avenue Bridge 905A, which embeds fabricated steel boxes into structural concrete band-beams.

Inverted U-beam – This refers to all U-beams that are not Rocla M-Lock Precsast Bridge U-beams.

Solid Slab Reinforced with Steel Beams – this is a concrete slab with cast-in beams, typically spare or re-purposed rail beams, not designed for composite action.

Pavement Type – Refer to Appendix A of the Detailed Visual Bridge Inspection Guidelines for Timber Bridges (Level 2 Inspections).

Surface Type – Refer to Appendix A of the Detailed Visual Bridge Inspection Guidelines for Timber Bridges (Level 2 Inspections).

Delineation post – typically a plastic post with reflective marker.

Visibility Barrier – A continuous barrier that does not offer appropriate performance under vehicle impact.

## 15.5 Procedure

The process for collecting and providing inventory information to Structures Engineering is managed through:

1) Complete the appropriate inventory form at a time when the design, including drafting, may be reasonably deemed to be complete and final. Inventory forms are available on the public Main Roads Western Australian website via Building Roads, Standard and Technical, Structures Engineering, Asset Management, Inventory Forms (link).

2) The Regional Asset Manager Structures shall ensure that these Attachments are promptly sent to StructEngReviews@mainroads.wa.gov.au.

## 15.6 Inventory Form Guidance

- General Bridge Inventory

  The General Bridge Inventory form shall be completed for all new structures, replacement structures and bridge refurbishments wherein either the bridge geometry or bridge type is changed.

- Sign Gantry Inventory

  The Sign Gantry Inventory Form shall be completed for any overhead structure spanning, or partially spanning (if cantilevered), a road carriageway for the specific purpose of carrying regulatory, advisory, warning, variable message (VMS) or directional sign.

- Culvert Inventory Information

  The Culvert Inventory Information form shall be completed for any culvert that replaces a numbered bridge.

- Tunnel Inventory Information

  The Section 15.4 definition of a tunnel should be used to confirm whether the Tunnel Inventory Information form or General Bridge Inventory form is applicable.

# SECTION 16 – HISTORICAL DESIGN AND LOAD RATING VEHICLES

This information is Part 16 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

Structures Asset Policy Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

## Authorisation

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**Adam Lim**
SENIOR ENGINEER STRUCTURES

Date: 18/08/2021

**Document No: 3912/02-16**

**SECTION 16**

## CONTENTS



REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
|---|---|---|---|
| All | 1 | 18/08/2021 | Review and update against AS5100 2017, Include historical vehicles cited by AS5100.7 2004, which are not included in AS5100 2017 |
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Custodian Endorsement

Jewely Parvin
Structures Asset Policy Engineer
Date: 18/08/2021

# 16 HISTORICAL DESIGN AND LOAD RATING VEHICLES

## 16.1 INTRODUCTION

This Section of the Manual shall be used to determine the load capacity of existing steel and concrete bridges in Western Australia (WA) from historical design vehicles and load rating vehicles.

The aim of load rating analysis is to determine the theoretical capacity of a bridge by calculating what proportion of different vehicles it can carry. These values are used for the assessment of heavy haulage movements throughout the State.

Section 16 has been prepared to provide details of historical vehicles that bridges may have been designed or load rated for that are no longer in current CODES or in Section 4 of this Manual. Comparisons can then be made between these vehicles and current rating vehicles to assist in the assessment of heavy vehicles. For more details, user needs to look at old codes.

## 16.2 HISTORICAL DESIGN VEHICLES

### 16.2.1 1965 Highway Bridge Design Specification (NAASRA)

- Type of Code – Working stress

Truck and Lane Loads

- Live Loads – Standard trucks or lane loads corresponding to truck trains. H loadings and H-S loadings (as below)
- Design Traffic Lane Width – 10-ft
- Number of design lanes

| Width between kerbs (feet) Excludes median widening and road curvature widening | Number of Design Traffic Lanes |
|---|---|
| over 20-30 incl. | 2 |
| over 30-42 incl. | 3 |
| over 42-54 incl. | 4 |
| over 54-66 incl. | 5 |
| over 66-78 incl. | 6 |
| over 78-90 incl. | 7 |
| over 90-102 incl. | 8 |
| over 102-114 incl. | 9 |
| over 114-126 incl. | 10 |

- Location – One vehicle per design lane applied anywhere on the structure
- Lane Reduction Factors –

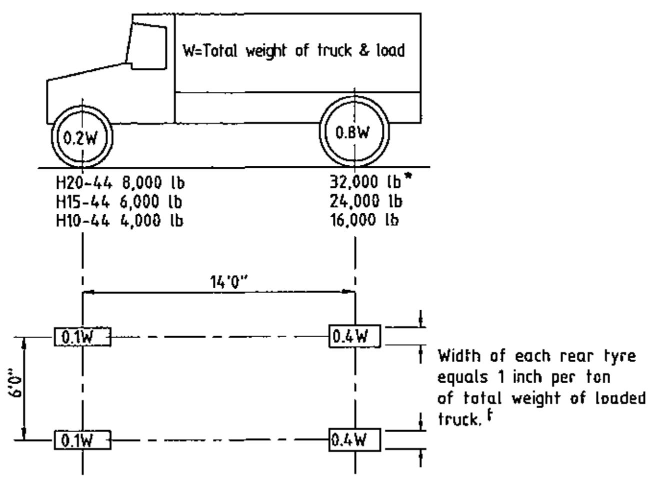
| | |
|---|---|
| 1or 2 lanes | 100% |
| 3 lanes | 90% |
| ≥4 lanes | 75% |

- Overload provision – Provision for overload shall be made by applying in any single lane an H or HS truck as specified, increased 100 per cent, and without concurrent loading of any other lanes. Combined dead, live and impact stresses resulting from such loading shall not be greater than 150 per cent of the allowable stresses allowed herein. The overload shall apply to all parts of the structure affected, including stringers, but expecting portions of the structure affected by individual wheel loads only.

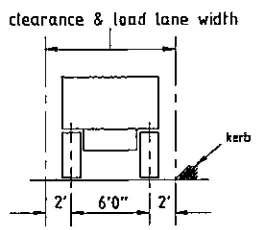
- Dynamic Load Allowance - $DLA = \frac{5000}{L+125}\,\%$ where L = span length in feet (more details are available in code for L)

Max. 30% and Min. 10%

Note: Not applicable to foundations, piles not rigidly connected to the superstructure, retaining walls, footways, timber structures and structures carrying more than 3ft of fill

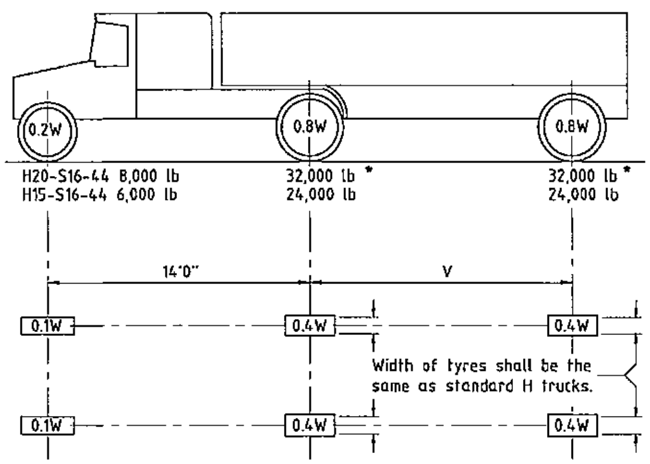



Length of contact of tyre with deck = 4" for front wheel.
8" for rear wheel.



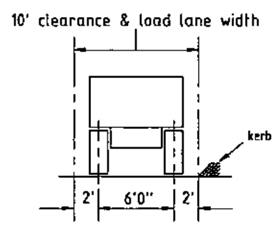

## STANDARD H TRUCKS

- * In the design of steel grid & timber decks only, for H20 loading, one axle load of 24,000 lb. or axle loads of 16,000 lb each spaced 4 ft. apart shall be used, whichever produces the greater stress, instead of the 32,000 lb. axle shown. Concrete slabs shall be designed for the 32,000 lb. axle.
- † Ton here means a weight of 2,000 lb.



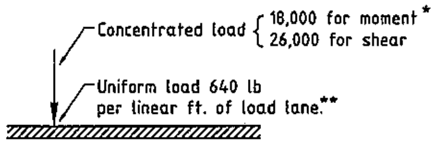

W= Combined weight on the first two axles which is the same as for the corresponding H truck.
V= Variable spacing - 14ft to 30ft, inclusive, spacing to be used is that which produces maximum stresses.

Length of contact of tyre with deck = 4" for front wheel.
8" for rear wheel.

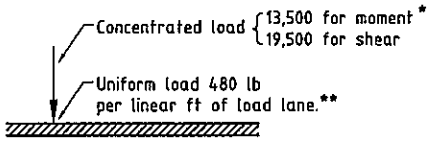



## STANDARD HS TRUCKS

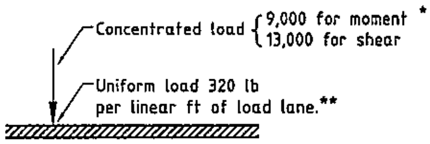
- * In the design of steel grid & timber decks only, for H20-S16 loading, one axle load of 24,000 lb. or two axle loads of 16,000 lb each spaced 4 ft. apart shall be used, whichever produces the greater stress, instead of the 32,000 lb. axle shown. Concrete slabs shall be designed for the 32,000 lb. axle.




H20-44 Loading
H20-S16-44 Loading




H15-44 Loading
H15-S12-44 Loading




H10-44 Loading

## H LANE & HS LANE LOADINGS

** To follow or precede, or be on both sides of the concentrated loads to produce the maximum stress.

* NOTE: For the loading of continuous spans involving lane loading, an additional concentrated load shall be placed in or other span in such a position as to produce maximum negative moment.

### 16.2.2 1970 Highway Bridge Design Specification (NAASRA)

No changes from 1965 Highway Bridge Design Specification (NAASRA)

### 16.2.3 1973 Highway Bridge Design Specification (NAASRA) Metric Version

- The content of the 1973 NAASRA Code is the same as the 1965 and 1970 versions except with minor numerical differences introduced during the conversion from imperial to metric. Note also that H and HS were renamed to M and MS.
- Type of Code – Working stress

Truck and Lane Loads

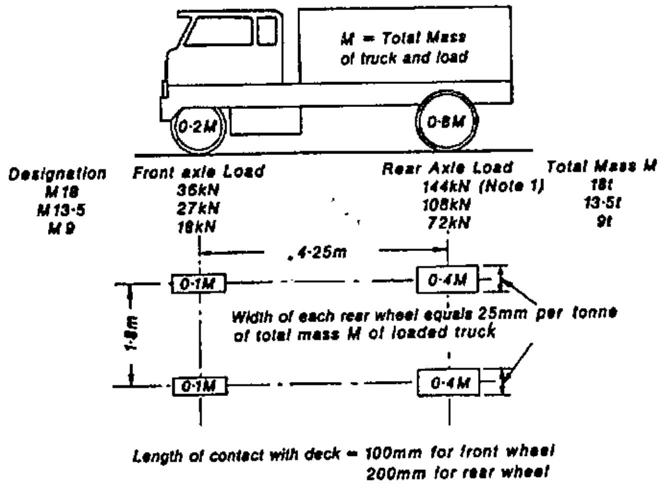
- Live Loads – Standard trucks or lane loads corresponding to truck trains. M loadings and MS loadings (as below)
- Design Traffic Lane Width – 3.0m
- Number of design lanes

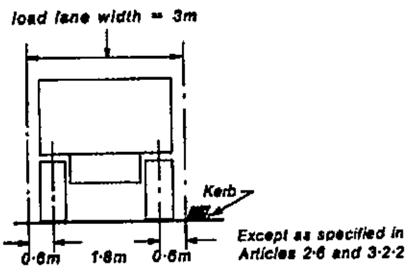
| Width between kerbs (metres)<br>Excludes median widening and road curvature widening | Number of Design Traffic Lanes |
|---|---|
| over 6 to 9 incl. | 2 |
| over 9 to 12.7 incl. | 3 |
| over 12.7 to 16.4 incl. | 4 |
| over 16.4 to 20.1 incl. | 5 |
| over 20.1 to 23.8 incl. | 6 |
| over 23.8 to 27.5 incl. | 7 |
| over 27.5 to 31.2 incl. | 8 |
| over 31.2 to 34.9 incl. | 9 |
| over 34.9 to 38.6 incl. | 10 |

- Location – One vehicle per design lane applied anywhere on the structure
- Lane Reduction Factors – 1or 2 lanes 100%
  3 lanes 90%
  ≥4 lanes 75%
- Overload provision – Provision for overload shall be made by applying in any single lane an H or HS truck as specified, increased 100 per cent, and without concurrent loading of any other lanes. Combined dead, live and impact stresses resulting from such loading shall not be greater than 150 per cent of the allowable stresses allowed herein. The overload shall apply to all parts of the structure affected, including stringers, but expecting portions of the structure affected by individual wheel loads only.
- Dynamic Load Allowance – $DLA = \frac{1600}{L+40}\,\%$ where L = span length in metre (More details are available in code for L)

  Max. 30% and Min. 10%

  Note: Not applicable to foundations, piles not rigidly connected to the superstructure, retaining walls, footways, timber structures and structures carrying more than 3ft of fill

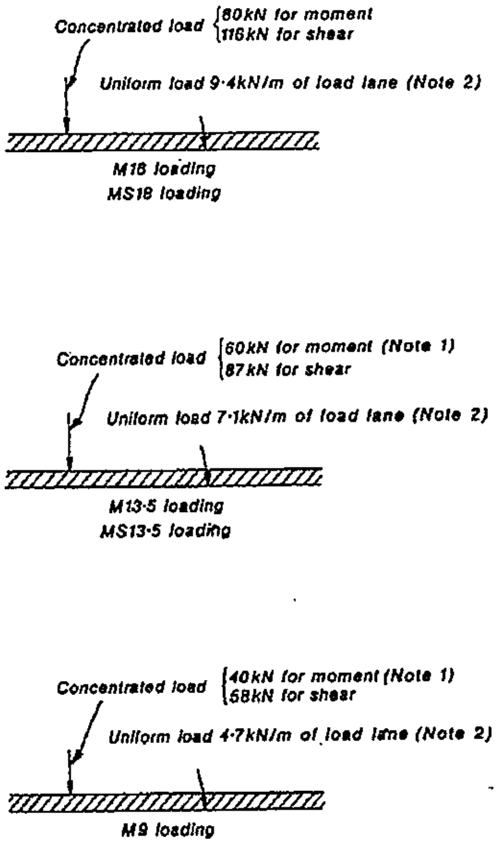


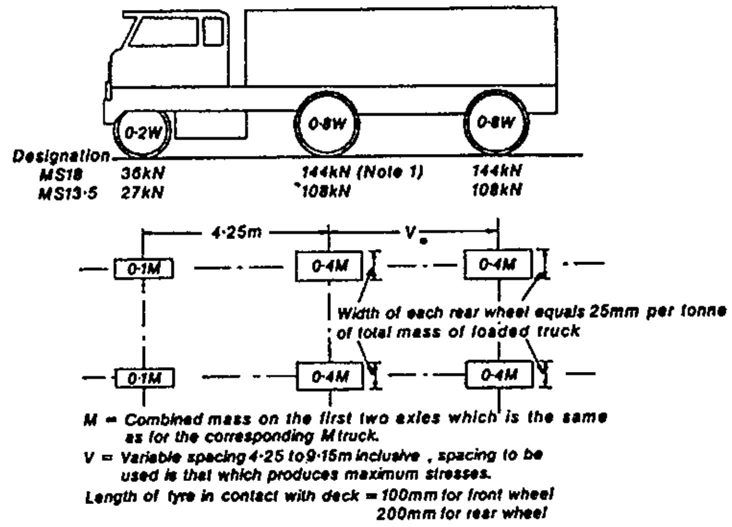



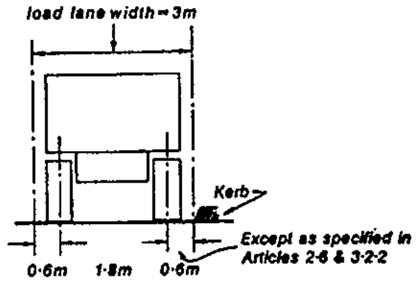

NOTES

1. In the design of steel grid and timber decks only, for M 18 loading, one axle load of 108 kN or two axle loads of 72 kN each spaced 1.2 m apart shall be used, whichever produces the greater stress, instead of the 144 kN axle shown.
All other deck systems shall be designed for the 144 kN axle. (The reason for the use of reduced loading on timber and steel grid decks is the ease and relative economy with which they can be replaced.)

## STANDARD M TRUCKS




NOTES

1. For the loading of continuous spans involving lane loading an additional concentrated load of equal weight shall be placed in one other span in such a position as to produce maximum negative moment.
2. To follow or precede, or be on both sides of the concentrated loads to produce the maximum stress.

## M LANE & MS LANE LOADINGS

NOTES

1. In the design of steel grid and timber decks only, for MS 18 loading, one axle load of 108 kN or two axle loads of 72 kN each spaced 1.2 m apart shall be used, whichever produces the greater stress, instead of the 144 kN axle shown.

All other deck systems shall be designed for the 144 kN axle. (The reason for the use of reduced loading on timber and steel grid decks is the ease and relative economy with which they can be replaced)

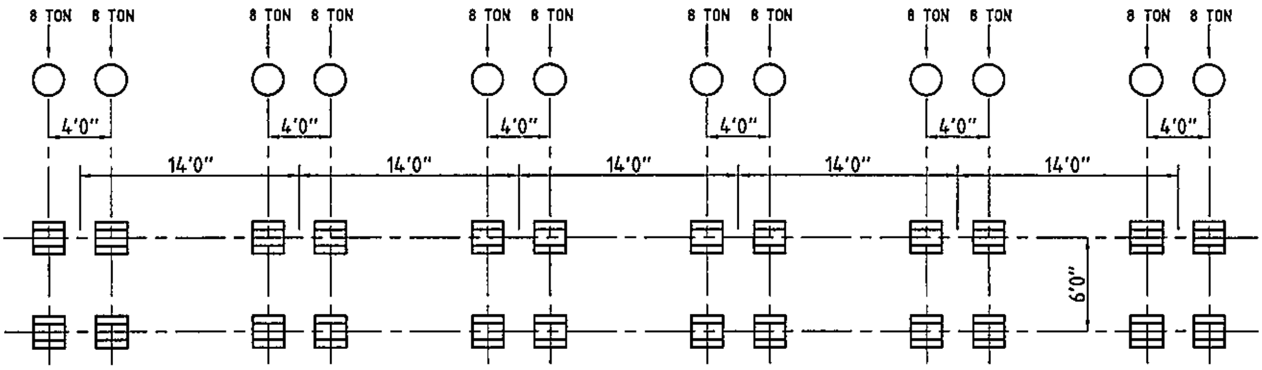
## STANDARD MS TRUCKS

### 16.2.4 1973 MRWA Road Train and Abnormal Vehicle

- Type of Code – Working stress

<u>Road Train</u>

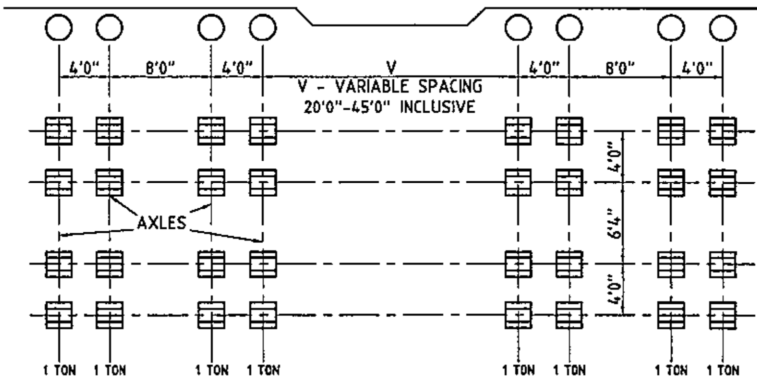
- Live Load – Road Train (as below)
- Design Lane Width – N/A
- Location – One vehicle per bridge applied anywhere on the structure without concurrent loading
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – $DLA = \frac{5000}{L+125}\%$ where L = span length in feet
  Max. 30% and Min. 10%




ROAD TRAIN

Abnormal Vehicle

- Live Load – Abnormal Load Vehicle (as below)
- Design Lane Width – N/A
- Location – One vehicle per bridge applied centrally without concurrent loading
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – DLA = 10%




25 UNITS FOR OTHER H+M ROADS.

30 UNITS FOR MAIN INDUSTRIAL ROADS IN METRO AREA.

ABNORMAL LOAD VEHICLE

(LOADED WITH 1 LOAD UNIT)

1 LOAD UNIT COMPRISES A LOAD OF 1 TON ON EA OF THE 8 AXLES = 8 TO TOTAL LOAD.

### 16.2.5 1976 NAASRA Bridge Design Specification

- Type of Code – Working stress

Vehicle Load for Spans Less than or Equal to 4m

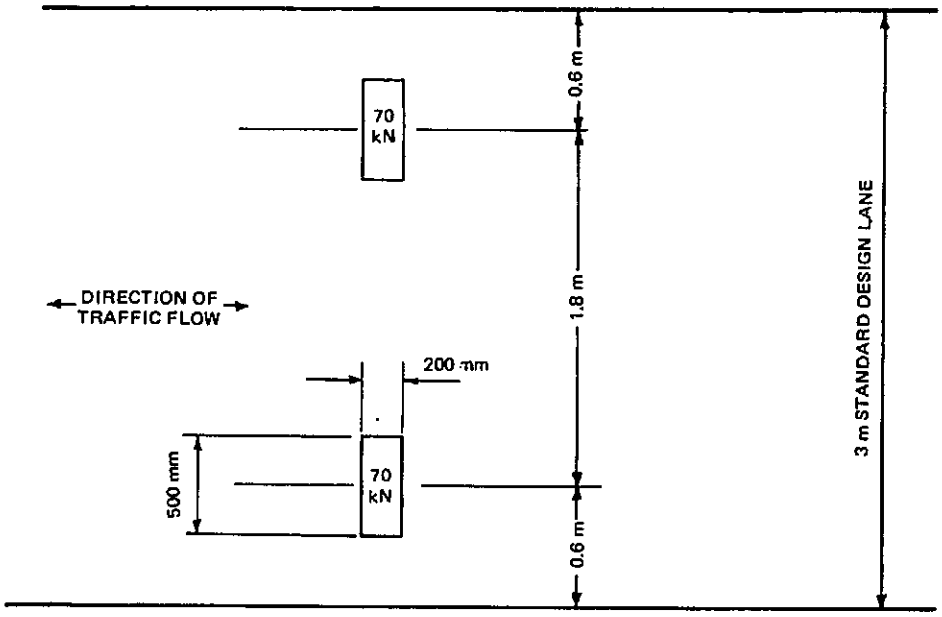
- Live Load - A14 Standard Vehicle Loading (as below)
- Design Traffic Lane Width – 3.0m
- Number of design lanes, n = width between kerbs / 3.1
- Location – One axle per design lane applied anywhere on the structure.
- Lane Reduction Factors –

| | |
|---|---|
| 1or 2 lanes | 100% |
| 3 lanes | 90% |
| ≥4 lanes | 75% |

- Dynamic Load Allowance – $DLA = \frac{1600}{L+40}\%$ where L = span length in metre (more details are available in code for L)

  Max. 30% and Min. 10%

Notes: DLA = 30% for culverts with cover ≤300mm and for cantilevers
= 20% for culverts with cover > 300mm and ≤600mm
= 10% for culverts with cover > 600mm and ≤1.0m
= 10% for Abnormal vehicles



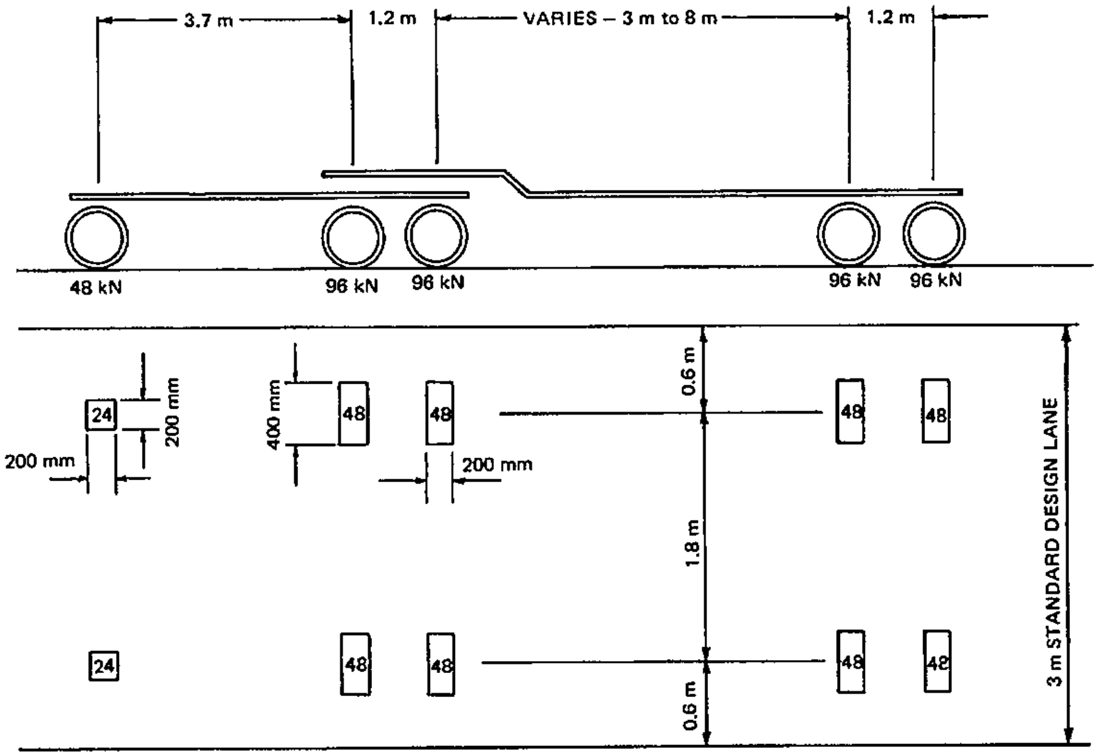

NOTES

1. Total mass on axle is 14.3 tonnes (figure shows loads).
2. In designing the deck system for local effects, the wheel nearest the kerb may be placed with its centre 0.3 m from the kerb face.
3. In continuous spans, for the calculation of maximum negative bending moment, an additional axle of similar weight and configuration shall be placed in the design lane so that the axle spacing is 4.25m

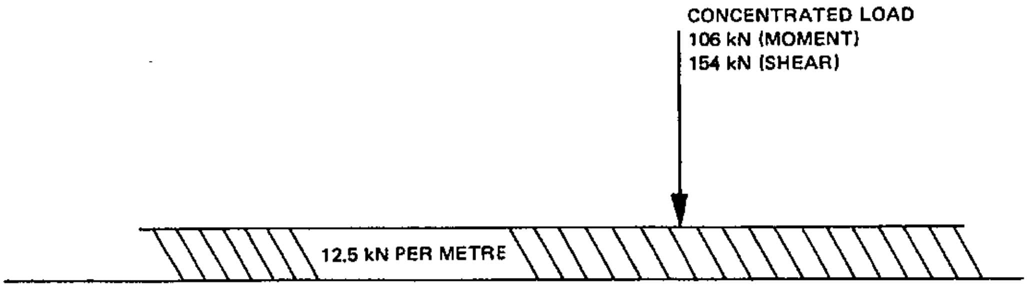
A14 STANDARD VEHICLE LOADING

Vehicle Load for Spans Equal to or Greater than 5m

- Live Load - T44 truck or lane loading (as below)
- Note: Lane loading applies to spans of 10m or greater only.
- Design Lane Width – As A14 above
- Location – One truck load or lane load per design lane applied anywhere on the structure. The lane loading shall be continuous or discontinuous.
- Lane Reduction Factors – As A14 above
- Dynamic Load Allowance – As A14 above




**T44 TRUCK LOADING**




**T44 LANE LOADING**

NOTES

1. Total mass of vehicle is ≤ 44 tonnes (figure shows loads).
2. For continuous spans the lane loading shall be continuous or discontinuous as may be necessary to produce maximum effects, and an additional concentrated load shall be placed in one other span in such a position as to produce maximum negative moment.
3. The T44 lane loading shall be considered as uniformly distributed over the width of the standard design lane.

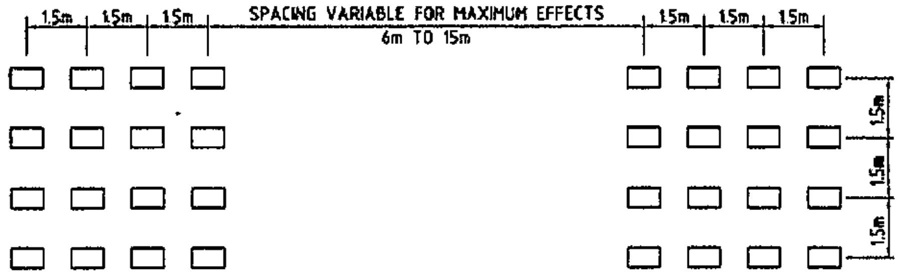
T44 STANDARD VEHICLE LOADING

## Vehicle Load for Spans Between 4 & 5m

- Live Load – Interpolate A14 & T44. Design effect = A14 + (T44 – A14)(span - 4)

Abnormal Load

- Live Load – Standard Abnormal Vehicle and MRWA 205 & 245 tonne variants (as below)
- Design Lane Width – N/A (loads not applicable for width between kerbs less than 6m)
- Location – One vehicle per bridge applied centrally without concurrent loading
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – DLA = 10%




STANDARD ABNORMAL VEHICLE
Plan of Wheel Positions

**NOTES**

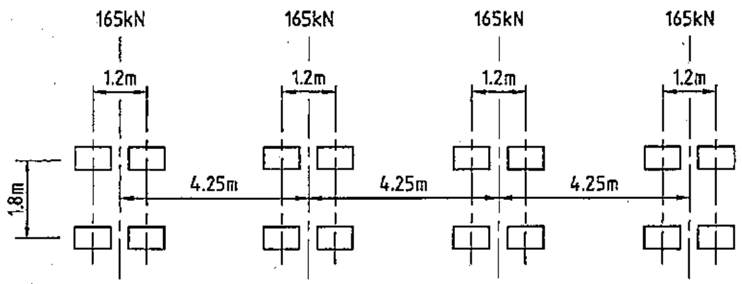
1. Load Per wheel = 60kN
2. Load Per axle = 240kN
3. Total mass of vehicle 196 Tonne (approx)

MRWA own increase to axle loads

205 Tonne = 251kN Per Axle
245 Tonne = 300kN Per Axle

### 16.2.6 1989 MRWA Bridge Design Manual Road Train

- Type of Code –Limit state
- Live Load – 4 or 6 x 16.5t Tandem Axles (as below)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – One vehicle per bridge applied anywhere on the structure without concurrent loading
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code




4 x 16.5t TANDEM AXLES

### 16.2.7 1992 Austroads Bridge Design Code

- Type of Code –Limit state

W7 Wheel Load

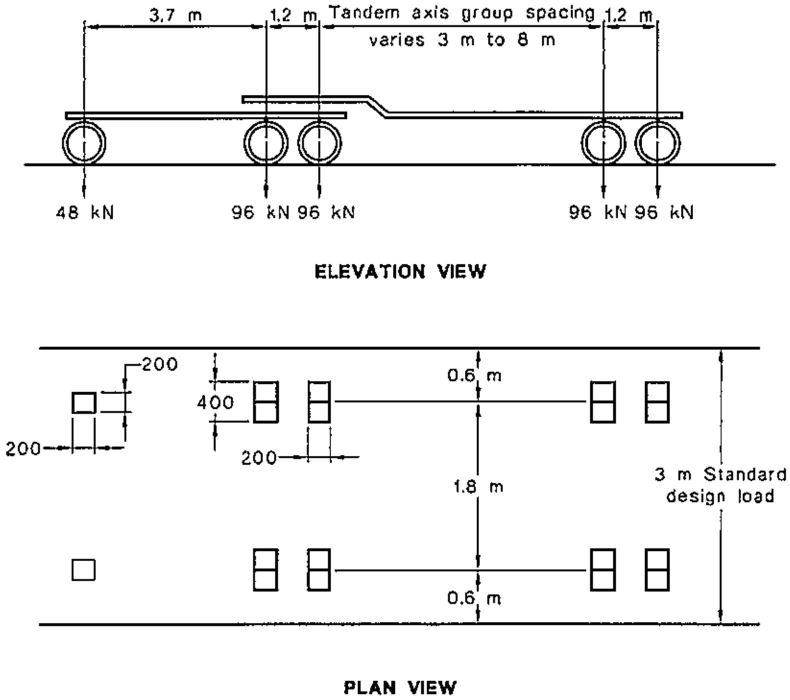
- Live Load - 70kN over a contact area of 500mm x 200mm
- Design Lane Width – N/A
- Ultimate Limit State Factor – 2.0
- Location – Anywhere on the bridge to produce the maximum local effect for the structural element under consideration for which the critical load is a single wheel load
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – DLA = 0.25 as minimum

T44 Truck Load and L44 Lane Load

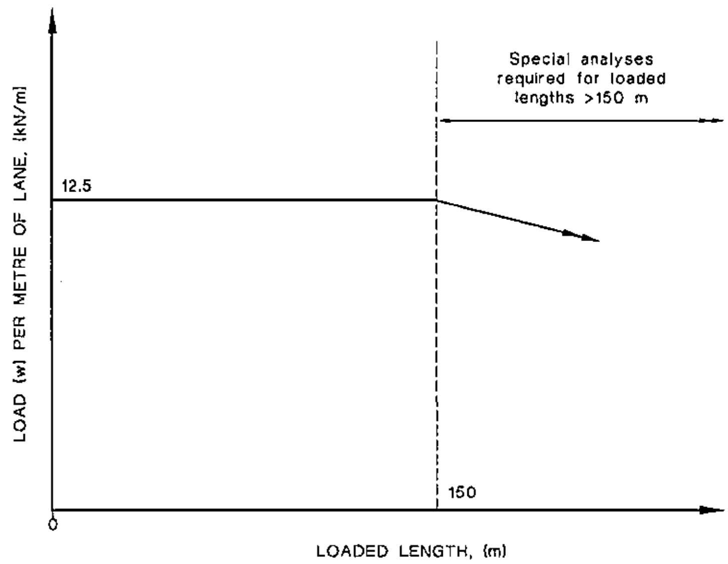
- Live Load - T44 truck or lane loading (as below)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – One truck load or lane load per design lane applied anywhere on the structure. The lane loading shall be continuous or discontinuous.
- Lane Reduction Factors –

| | |
|---|---|
| 1 lane | 1.0 |
| 2 lanes | 0.9 |
| 3 lanes | 0.8 |
| 4 lanes | 0.7 |
| 5 lanes | 0.6 |
| ≥6 lanes | 0.55 |

- Dynamic Load Allowance (as below)



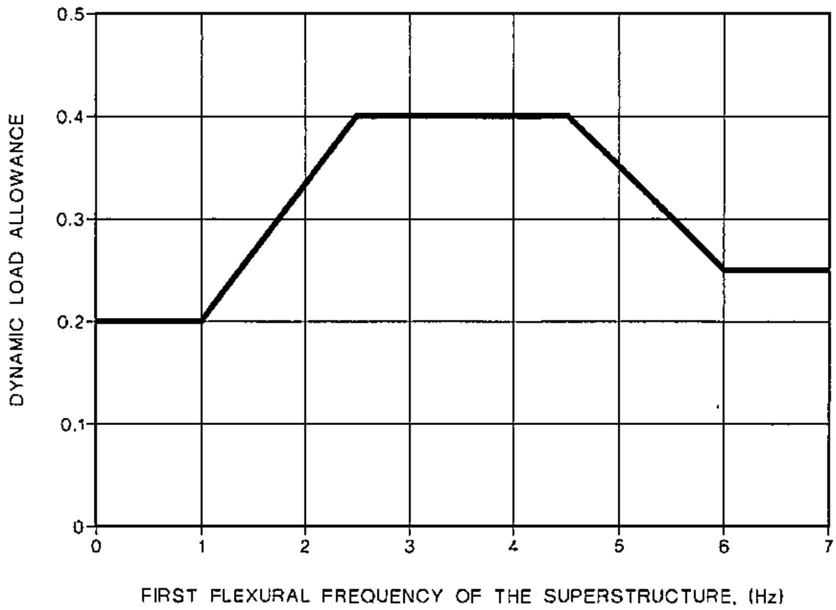

## T44 TRUCK LOAD




NOTES

1. L44 Lane loading shall consist of a uniformly distributed load as shown above together with a concentrated load of 150kN
2. For continuous spans the lane loading shall be continuous or discontinuous as may be necessary to produce maximum effects, and an additional concentrated load shall be placed in one other span in such a position as to produce maximum negative moment.
3. The L44 lane load does not apply to spans less than 10m.

## L44 LANE LOAD
## (Uniformly distributed part only)



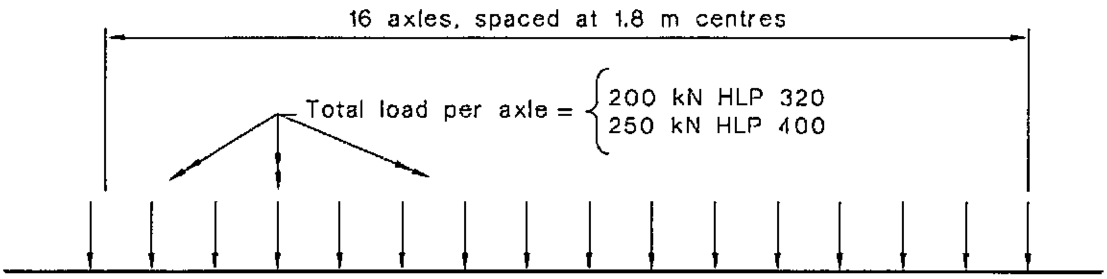

NOTE: A range of first flexural frequencies from 0.9 to 1.1 times the calculated superstructure frequency should be considered. The dynamic load allowance adopted should be the maximum value obtained from Figure A4 for this frequency range.

DYNAMIC LOAD ALLOWANCE FOR T44 TRUCK AND L44 LANE LOADS (ONE LANE LOADED)

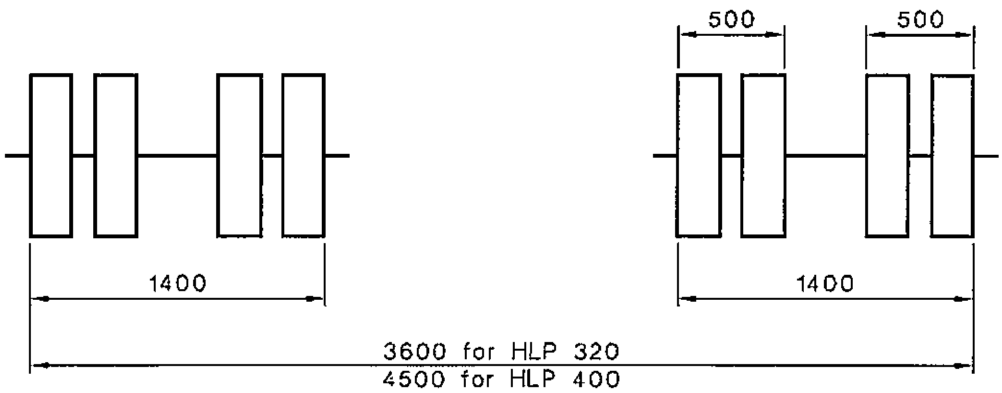
Note: DLA to be taken as full at ground line, reducing linearly to zero at depth 2m. For buried structures this is 0.4 reducing to 0.1.

HLP320 and HLP400 Platform Loads

- Live Load – HLP320 or HLP 400 platform loading (as below)
- Design Lane Width – N/A
- Ultimate Limit State Factor – 1.5
- Location – One platform per bridge along the centreline ±1m of the bridge without concurrent loading (undivided) or along the centreline ±1m of the carriageway in the travel direction with 50% T44 truck loading or L44 lane loading on the other carriageway.
- Lane Reduction Factors – N/A
- Dynamic Load Allowance - DLA = 0.1




**ELEVATION VIEW**




**END VIEW OF AN HLP AXLE**

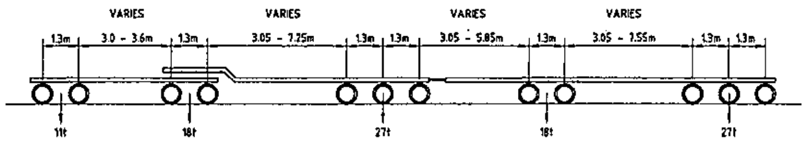
FOR HLP LOADS LATERAL SPACING OF DUAL WHEELS ALONG AN AXLE

Note: For continuous bridges the loading may be separated into two groups with a central gap of between 6 m and 15 m, the gap being chosen to give the most adverse effect.

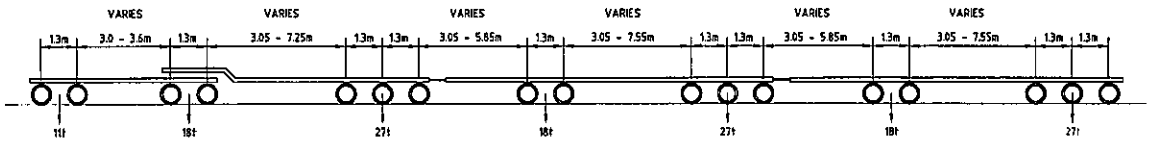
### 16.2.8 1998 MRWA Bridge Branch Design Information Manual Standard Road Trains

- Type of Code –Limit state
- Live Loads – 101t Double Bottom Road Train and 146t Triple Bottom Road Train (as below)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – As per T44 of 1992 Austroads Bridge Design Code
- Lane Reduction Factors – As per T44 of 1992 Austroads Bridge Design Code
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code

DOUBLE BOTTOMED DESIGN VEHICLE
(G.C.M = 101 tonnes)

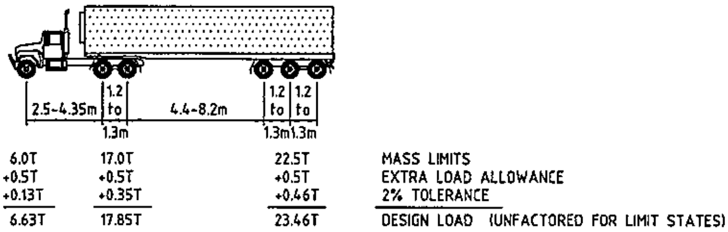



TRIPLE BOTTOMED DESIGN VEHICLE
(G.C.M = 146 tonnes)

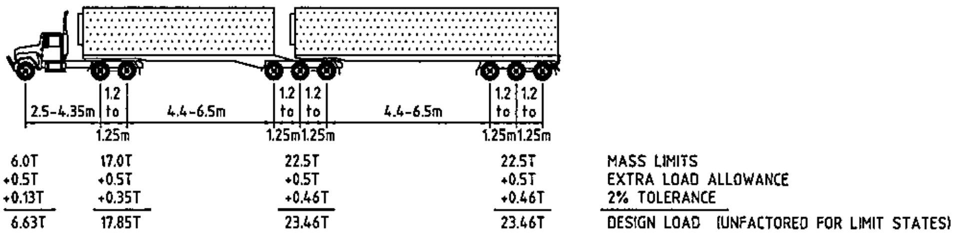



### 16.2.9 1999 MRWA NRTC Vehicles

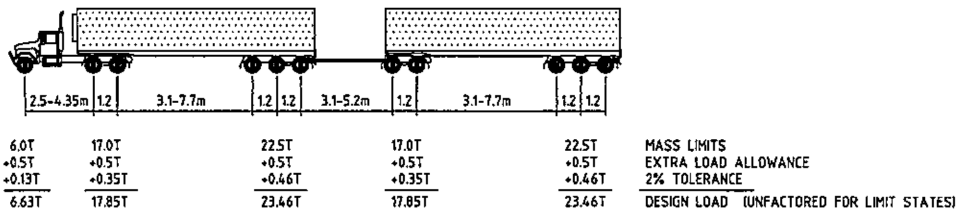
- Type of Code –Limit state
- Live Loads – Semi Trailer, Tri-Tri B-Double, Double Bottom Road Train and Triple Bottom Road Train at road friendly suspension weights (as below)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – As per T44 of 1992 Austroads Bridge Design Code
- Lane Reduction Factors – As per T44 of 1992 Austroads Bridge Design Code
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code



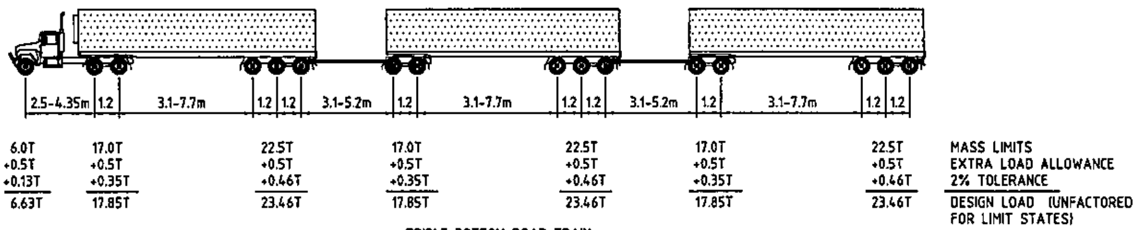

SEMI - TRAILER
GROSS COMBINED MASS 45.5T




TRI - TRI, B - DOUBLE
GROSS COMBINED MASS 68T




DOUBLE BOTTOM ROAD TRAIN
GROSS COMBINED MASS 85T




TRIPLE BOTTOM ROAD TRAIN
GROSS COMBINED MASS 124.5T

### 16.2.10 1999 MRWA Interim Bridge Design Loads

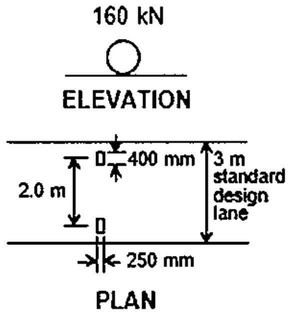
- Type of Code –Limit state

Wheel Load

- Live Load – W80 as per AS5100.2 Clause 7.2.2
- Design Lane Width – N/A
- Ultimate Limit State Factor – As per AS5100.2 Clause 7.10
- Location – As per AS5100.2 Clause 7.2.2
- Lane Reduction Factors – N/A
- Dynamic Load Allowance - DLA = 30%
  Note: DLA to be taken as full at ground line, reducing linearly to zero at depth 2m. For buried structures this is full at ground line reducing to 0.1.

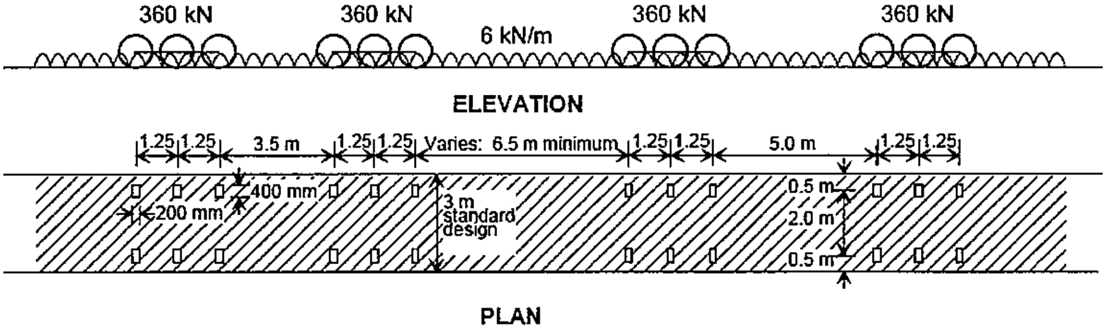
Axle Load

- Live Load – A160 axle load (as below)
- Design Lane Width – 3.0m
- Number of design lanes, n = width between barriers / 3.1
- Ultimate Limit State Factor – As per AS5100.2 Clause 7.10
- Location – One axle load per design lane (as below) anywhere on the structure
- Lane Reduction Factors – As per AS5100 Clause 7.6
- Dynamic Load Allowance - DLA = 30% (note: AS5100-2004 & 2017 are 40%)
  Note: DLA to be taken as full at ground line, reducing linearly to zero at depth 2m. For buried structures this is full at ground line reducing to 0.1.



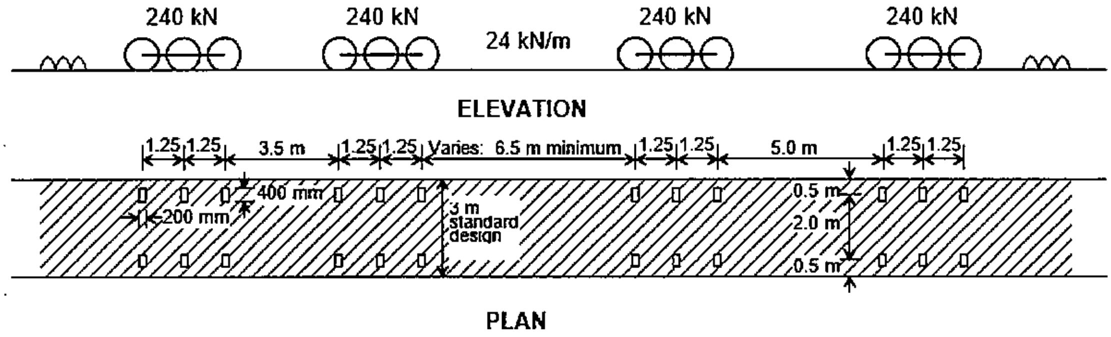

Moving Traffic Load

- Live Load – M1600 moving traffic load (as below. Note this is different to AS5100)
- Design Lane Width – 3.0m
- Number of design lanes, n = width between barriers / 3.1
- Ultimate Limit State Factor – AS5100.2 Clause 7.10
- Location – As per AS5100.2 Clause 7.2.4
- Lane Reduction Factors – As per AS5100.2 Clause 7.6
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code
  Note: DLA to be taken as full at ground line, reducing linearly to zero at depth 2m. For buried structures this is full at ground line reducing to 0.1.



Stationary Traffic Load

- Live Load – S1600 stationary traffic load (as below. Note this is different to AS5100)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – As per AS5100.2 Clause 7.10
- Location – As per AS5100.2 Clause 7.2.5
- Lane Reduction Factors – As per AS5100.2 Clause 7.6
- Dynamic Load Allowance – DLA = 0%



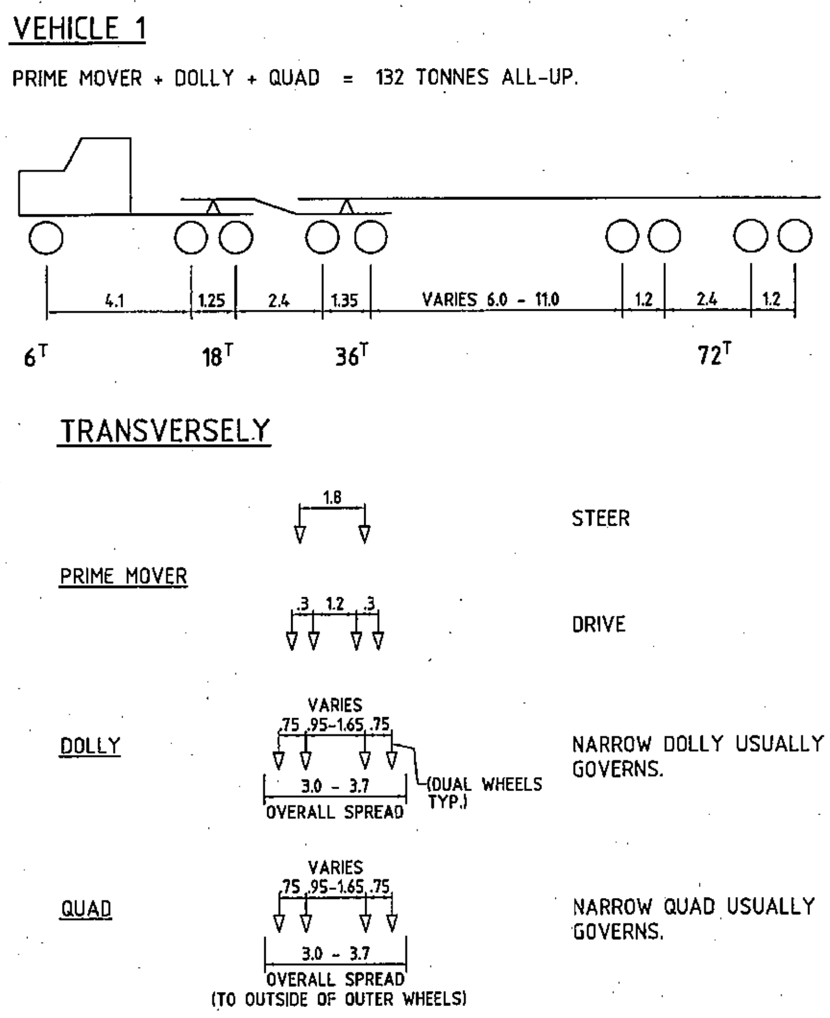

### 16.2.11 2004 Australia Standard Bridge Design AS5100

- Type of Code –Limit state
- W80, A160, M1600, S1600, HLP320 and HLP400 as per AS5100.2 2017

## 16.3 HISTORICAL LOAD RATING VEHICLES

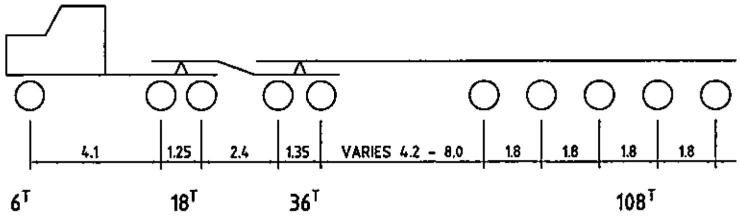
### 16.3.1 1993 MRWA Bridge Branch Design Information Manual

- Type of Code –Limit state
- Live Loads – Rating Vehicles 1-4 (as below)
- Design Lane Width – N/A
- Ultimate Limit State Factor – 1.5
- Location – One vehicle for single carriageway bridges ± 1m of the bridge centreline without concurrent loading
  One Vehicle 1-2 for dual carriageway bridges ± 1m of the carriageway centreline with 50% T44/L44 on the other carriageway
  One Vehicle 3-4 for dual carriageway bridges ± 1m of the carriageway centreline without concurrent loading
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – DLA = 10%

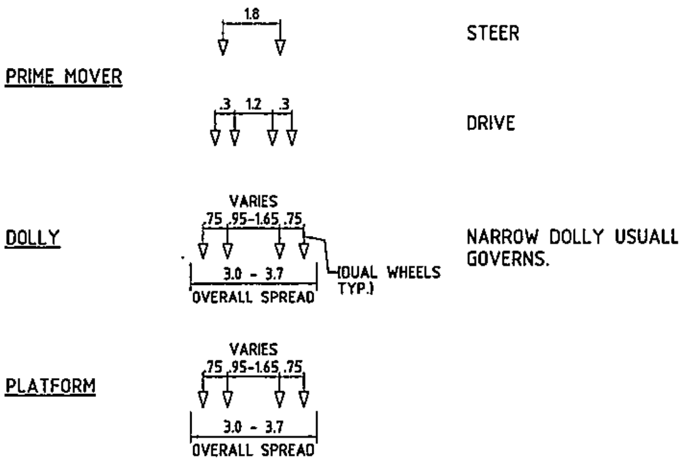


## VEHICLE 2

PRIME MOVER + DOLLY + 6 LINE PLATFORM = 168 TONNES ALL-UP.

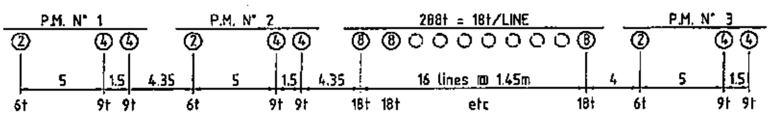



## TRANSVERSELY

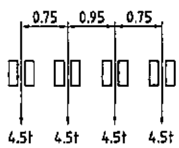



## VEHICLE 3

16 LINE 3m WIDE PLATFORM WITH LINE SPACING AT 1.45m. (288 TONNES GROSS) PLUS 3 PRIME MOVERS. (2 AT FRONT AND 1 AT REAR)



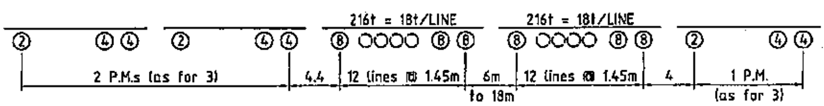

(P.M. = PRIME MOVER)




TRANSVERSE
(3m WIDE)

## VEHICLE 4

2/12 LINE PLATFORMS AT 1.45m. (192 WHEELS)
PLUS 3 PRIME MOVERS



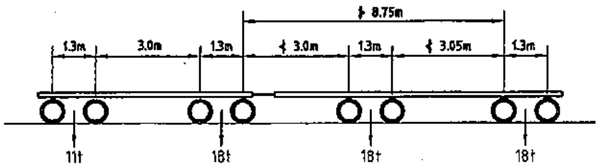

(P.M. = PRIME MOVER)

TRANSVERSE AS FOR 3.

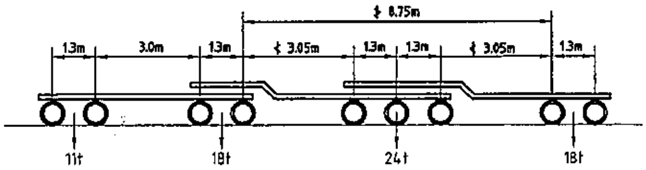
### 16.3.2 1993 MRWA Load Rating Vehicles

- Type of Code –Limit state
- Live Loads – Vehicles 1-8 representing various semi trailers, B-doubles and road trains (as below)
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – As per T44 of 1992 Austroads Bridge Design Code
- Lane Reduction Factors – As per T44 of 1992 Austroads Bridge Design Code
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code

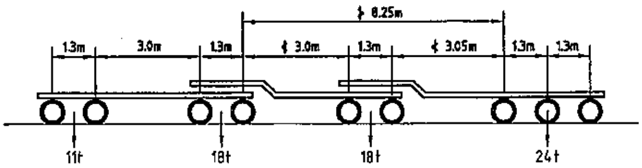
VEHICLE 1: TRUCK PLUS 4 AXLE DOG TRAILER
(G.C.M = 65 tonnes)



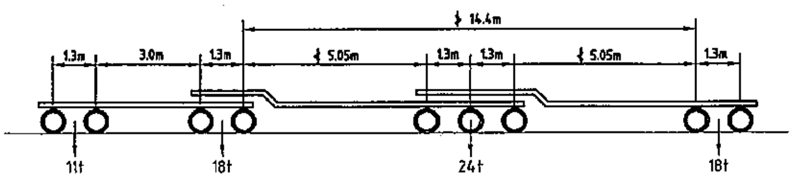

VEHICLE 2: SHORT "B" DOUBLE TRI-TANDEM
(G.C.M = 71 tonnes)



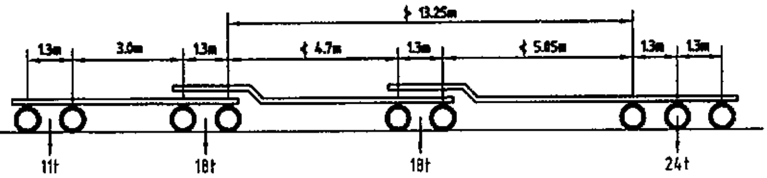

VEHICLE 3: SHORT "B" DOUBLE TANDEM-TRI
(G.C.M = 71 tonnes)



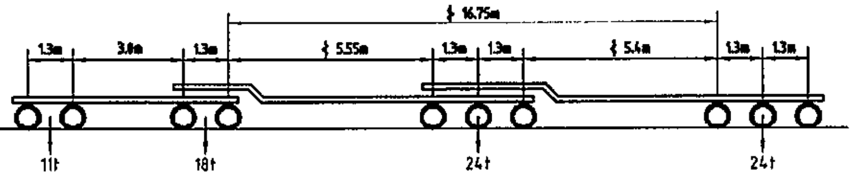

VEHICLE 4: LONG "B" DOUBLE TRI-TANDEM
(G.C.M = 71 tonnes)



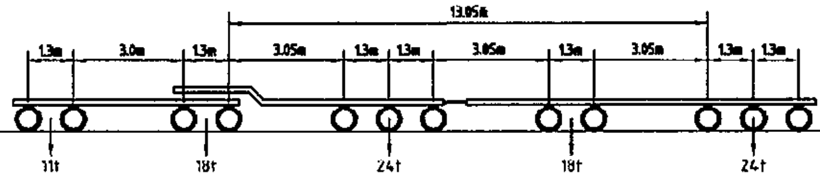
**VEHICLE 5:** LONG "B" DOUBLE TANDEM-TRI
(G.C.M = 71 tonnes)



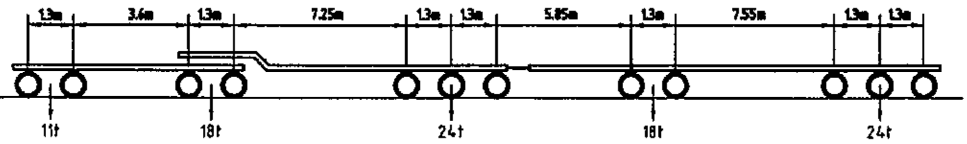

**VEHICLE 6:** TRI-TRI "B" DOUBLE
(G.C.M = 77 tonnes)




**VEHICLE 7:** SHORT DOUBLE BOTTOM ROAD TRAIN
(G.C.M = 95 tonnes)




**VEHICLE 8:** LONG DOUBLE BOTTOM ROAD TRAIN
(G.C.M = 95 tonnes)



### 16.3.3 1995 MRWA Bridge Branch Design Information Manual

- Type of Code –Limit state

Group 1 Vehicles

- Live Load – Group 1 Vehicles 1-3 as per the same vehicles in Section 4 of this Manual
- Design Lane Width – 3.0m
- Ultimate Limit State Factor – 2.0
- Location – As per T44 of 1992 Austroads Bridge Design Code , but only one rating vehicle to be on the bridge with T44/L44 in the other lane(s) positioned to give the worst effects
- Lane Reduction Factors – As per T44 of 1992 Austroads Bridge Design Code
- Dynamic Load Allowance – As per T44 of 1992 Austroads Bridge Design Code , refer AS5100.7 Clause A2.2.10

Group 2 Vehicles

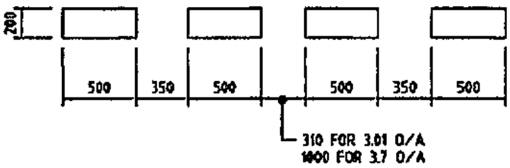
- Live Loads – Group 2 Vehicles 1-6 (as below)

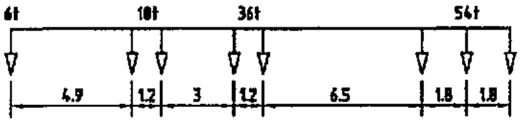
  Note: Group 2 Vehicles 1-2 and 5 are identical to the same vehicles in Section 4 of this Manual and Group 2 Vehicle 4 is identical in configuration to the same vehicle in Section 4 of this Manual but was considered at both vehicle platform spreads
- Design Lane Width – N/A
- Ultimate Limit State Factor – 1.5
- Location – One vehicle for single carriageway bridges ± 1m of the bridge centreline without concurrent loading

  One vehicle for dual carriageway bridges ± 1m of the carriageway centreline with T44/L44 on the other carriageway
- Lane Reduction Factors – N/A
- Dynamic Load Allowance – DLA = 10%

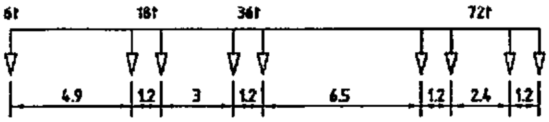
**NOTE**

VEHICLES 1-4 CAN HAVE AN O/A WIDTH OF EITHER 3.01m OR 3.7m.
VEHICLE 5 IS 3.01m O/A
VEHICLE 6 IS 3.7m O/A

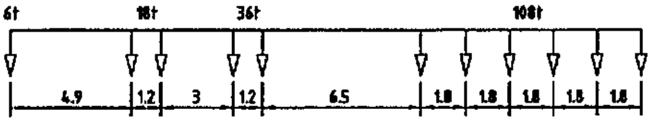


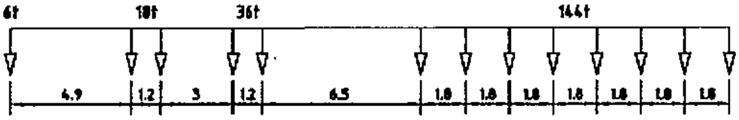

1. PRIME MOVER + 36 TONNE DOLLY + 54 TONNE TRIAXLE
(G.C.M. = 114 TONNES)



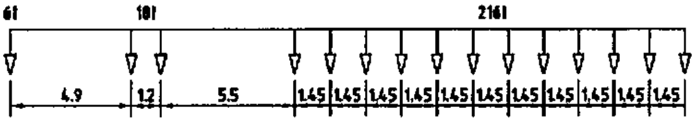

2. PRIME MOVER + 36 TONNE DOLLY + 72 TONNE SPREAD QUAD
(G.C.M. = 132 TONNES)



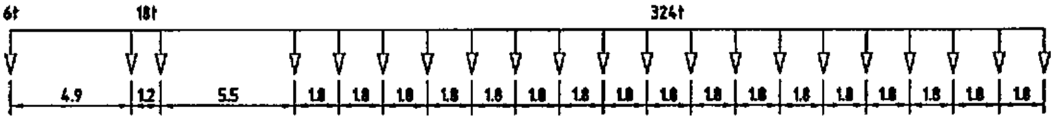

3. PRIME MOVER + 36 TONNE DOLLY + 108 TONNE 6-LINE PLATFORM
(G.C.M. = 168 TONNES)




4. PRIME MOVER + 36 TONNE DOLLY + 144 TONNE 8-LINE PLATFORM
(G.C.M. = 204 TONNES)




5. PRIME MOVER + 216 TONNE 12-LINE PLATFORM
(G.C.M. = 240 TONNES)




6. PRIME MOVER + 324 TONNE 18-LINE PLATFORM
(G.C.M. = 348 TONNES)

## GROUP 2 RATING VEHICLES

# SECTION 17 – EXTERNAL AUTHORITIES

This information is Part 17 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

The Structures Design & Standards Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

**Authorisation**

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

**A LIM**
SENIOR ENGINEER STRUCTURES

Date: 16/04/18

**Document No: 3912/02-17**

**SECTION 17**

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REVISION STATUS

| Page No. | Rev. No. | Rev. Date | Revision Description |
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| All | 1 | 15/12/05 | Complete review |
| All | 2 | 13/04/18 | Complete review |
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Custodian Endorsement

M RAJAKARUNA
Structures Design & Standards Engineer
Date: 13/04/18

# 17 LIST OF EXTERNAL AUTHORITIES

Before the commencement of a design project it is necessary to contact ALL known appropriate external authorities that may be affected. This applies mostly to proposed works but is nevertheless appropriate to maintenance works, particularly where pile driving or temporary works are likely to cause disruption to the immediate environment, community or traffic.

This list records various known authorities with comments on area of authority, responsibility or interest. It should be noted that this list does not include Main Roads Regions, Mining Companies, Landowners, etc.

Different authorities will need to be contacted for different jobs. It is the responsibility of the Designer to check that all authorities likely to be affected by a project are contacted.

It should not be assumed that this list is comprehensive or final.

| AUTHORITIES | AREA OF RESPONSIBILITY OR INTEREST |
|---|---|
| Department of Agriculture and Water Resources | Matters likely to impact land in an agricultural sense, e.g. backwater, salinity, waterway diversion of realignment. |
| Department of Primary Industries and Regional Development | Initial contact via Main Roads. |
| Department of Planning, Lands and Heritage | Manages the location of Aboriginal sites of significance and matters concerning social and economic equity for Indigenous people, respect for the land or unique heritage and culture. See also Local Aboriginal Groups.<br><br>Includes listed structures of historical significance. See also Australian Heritage Council. |
| Australian Heritage Council | Matters concerning significant historical sites where Federal funding is involved. |
| Local Aboriginal Groups | Matters concerning social and economic equity for Indigenous people, respect for the land or unique heritage and culture. |
| Independent Regional Development, eg:<br>• Regional Development Council<br>• Western Australian Regional Development Trust<br>• Gascoyne Development Commission (DC)<br>• Goldfields DC<br>• Great Southern DC<br>• Kimberley DC<br>• Mid West DC<br>• Peel DC<br>• Pilbara DC<br>• South West DC<br>• Wheatbelt DC | Independent Partners to the Department of Primary Industries and Regional Development. |
| Department of Transport – Cycling | Cycle path requirements. |
| WestCycle | Peak cycling body for WA |
| Department of the Environment and Energy | Designs and implements Australian Government policy and programs to protect and conserve the environment, water and heritage, promote climate action, and provide adequate, reliable and affordable energy. |
| Conservation Council of WA | Matters relating to environmentally sensitive areas. Matters relating to Wetlands Conservation Society will be identified by Conservation Council. |

| | |
|---|---|
| Department of Biodiversity, Conservation and Attractions | Matters relating to works likely to have an environmental<br>impact re-National Parks, State Forests, Marine Parks, Conservation Parks, dieback control, flora and fauna etc, the Swan and Canning Rivers / tributaries and adjoining land. |
| Department of Fire & Emergency Services<br>• Fire and Rescue Service<br>• St. John Ambulance<br>• Police Service WA | All emergency services to be advised of intent to close bridge/road. To be done at design stage in case of access or staging problems. |
| Department of Water and Environmental Regulation | The DWER conducts environmental impact assessments and develops policies to protect the environment. The DWER also monitors compliance with the conditions of Ministerial Statements.<br><br>Responsible for the waters and associated land within declared management areas. Must approve any alteration to the bed or banks of waterways within declared management areas of the State. |
| Landgate | Responsible for land tenure throughout WA. For mining tenements see Department of Industry, Innovation and Science. |
| Local Government Authority | Road or bridge closures, scheme approval, drainage methods, footpath requirements, services etc. |
| Department of Transport | Preparation of land use strategies and development. Responsible to ensure planned development is in accordance with sound planning principles. Initial contact via Main Roads. |
| Department of Transport (Marine) | Responsible to ensure that any navigable body of water is not modified in any way likely to create a navigation hazard. Jurisdiction includes recreational waters set aside for water skiing, sailing etc. |
| Department of Mines, Industry Regulation and Safety | Geological and hydrological advice such as site investigation, water bores, quarries, road cuttings etc. |
| National Trust (WA) | Details of National Estates. Copy of National Estates Register held in Main Roads Library. No statutory requirement for contact, however where historical sites are involved, contact is advised as a courtesy. |

| | |
|---|---|
| Service providers, eg:<br>• Alinta Energy<br>• Atco Gas<br>• BP (Oil pipeline)<br>• Natural Gas Pipelines<br>• Telecommunication Groups (Telstra, Optus, etc)<br>• Water (Water Corporation of WA, Harvey Water, Aqwest (Bunbury), Busselton Water Board)<br>• Western Power/Horizon Power | Mandatory contact required for possible relocation of existing service or requirement for new service. |
| Public Transport Authority (PTA) [bus; passenger rail] | Bus servicing considerations for engineering design, scheduling and/or disruptions to services.<br>Mandatory contact for all passenger rail impacts and interfaces. |
| Arc Infrastructure/Aurizon [freight rail] | Mandatory contact for all freight rail impacts and interfaces. |
| Private Rail Operators, eg:<br>• Rio Tinto<br>• BHP | Mandatory contact for all private rail impacts and interfaces. |
| WA Farmers Federation | Mandatory contact required for all proposed new bridge construction in the rural areas of the State. |
| Other Environmental and Resource Groups, eg:<br>• Landcare District Committees<br>• Pastoralists and Graziers Association<br>• Environs Kimberley<br>• North Metro Conservation Group<br>• Northern Agricultural Catchment Council<br>• Perth Region NRM<br>• Rangelands Natural Resource Management WA<br>• South Coast National Resource Management Inc.<br>• South West Catchment Council<br>• Urban Bushland Council | Various non-government groups.<br><br>Landcare District Committees - found throughout the State, refer Department of Agriculture and Water Resources.<br><br>Pastoralists & Graziers Association - mostly Northern Areas of the State. |

# SECTION 18 – PREPARATION OF DESIGN REPORT (OCCUPATIONAL SAFETY AND HEALTH)

This information is Part 18 of the Bridge Branch Design Information Manual and is owned and controlled by the Senior Engineer Structures.

Senior Design Engineer is the delegated custodian. All comments and requests for changes should be submitted to the delegated custodian.

## Authorisation

As head of Structures Engineering of Main Roads Western Australia, I authorise this issue and the use of this Information.

R J Scanlon

R SCANLON
SENIOR ENGINEER STRUCTURES

Date: 16 June 2009

Document No.: 3912/02-18

**Controlled Copies shall be marked accordingly**

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Revision Authorisation

R SCANLON
Senior Engineer Structures
Date:

# 18 PREPARATION OF DESIGN REPORT (OCCUPATIONAL SAFETY AND HEALTH)

## 18.1 Introduction

This Section of the Manual describes the mandatory requirements for Designers to provide clients with a written report on the occupational safety and health (OSH) aspects of their designs. In addition it provides guidance to Designers on a basis for identifying the types and extent of hazards and risks to be addressed.

## 18.2 Background

Regulations relating to the National Standard for Construction Work came into operation for the civil/commercial construction sector on 3$^{rd}$ January 2008.

These regulations introduce requirements relating to the provision of information, consultation, planning, documentation and other measures to ensure occupational safety and health in the building and construction industry.

These regulations are contained in Division 12 of Part 3 of the Occupational Safety and Health Regulations 1996.

The regulations apply to main contractors and people with control of construction work, clients commissioning design and construction work and designers doing design work for construction projects. This manual covers design requirements only.

## 18.3 Designer's Requirements

Designers must provide their clients with a written report on the occupational safety and health aspects of their designs. The client must ensure that this information is passed on to the main contractor and to anyone who obtains the end product of the construction work from the client.

## 18.4 Design Report

### *18.4.1 Requirements under the Regulations*

In accordance with Regulation 3.140 of Division 12 of Part 3 of the Occupational Safety and Health Regulations 1996, the Designer's OSH report must identify the following aspects of the design for the constructor to consider:

- The hazards associated with the construction work required to build the design, (for example, hazardous structural features, hazardous construction materials or hazardous procedures or practices);
- The designer's assessment of the risk of injury or harm resulting from those hazards;
- The action the designer has taken to reduce those risks, (for example, changes to the design or changes to construction methods or construction materials); and
- Any parts of the design where the hazards have been identified but not resolved.

### 18.4.2 Preparation of the Report

The extent and type of hazards are only broadly defined within the Regulations. Given that a construction site is potentially a hazardous environment due to the nature of the work, the Designers might elect to include all hazards associated with the work site and prepare mitigation actions accordingly.

Structures Engineering considers that in most cases this would result in a report that had a level of detail that would be a duplication of other processes whilst not adding value in achieving the objectives of the regulations. One reason for this is that MRWA has a pre-qualification system for contractors that ensures the constructor is qualified and experienced to undertake the construction works. In addition, the contractor is required within the contract to prepare a range of management plans that include for site safety.

Therefore in preparing the Report the Designer may assume that the constructor is an experienced builder, unless it is known to the contrary, and identify only those hazards that may be of a non-standard nature, unusual, specific to the design or otherwise noteworthy. Some examples are given in Appendix B to illustrate the types of design specific hazards that should be included in the Report. Other hazards that are typically found on construction sites such as traffic, working at heights or with machinery and tools may be considered to be of a standard nature, familiar to the Contractor and within its responsibility.

# APPENDIX A - PROCEDURE FOR DEVELOPMENT OF THE REPORT

1. The Designer during the design stage must assess the following:

   - Is the design complex with potential hazards associated with the complexity of the design impacting on the level of construction hazard?
   - Are there any designer identified construction hazards associated with the site conditions, features, topography, site constraints etc.?
   - Is the design innovative or unusual, requiring construction methodology, techniques and equipment which may not be familiar to an appropriately pre-qualified constructor?
   - Is the design straightforward, but with certain hazards associated with the construction?
   - Are there any specific or unusual interfaces with other agencies that might constitute an unusual hazard during construction?

2. Based on the above assessments, the Designer must identify and document construction activities and materials that are potentially hazardous. It is considered that designs of standard type structures which are regularly constructed would require minimum input into a Report prepared by the Designer, compared to more complex or innovative designs.

3. The next step is to assess and rate the risk associated with the identified potential hazard. Refer to the 'Corporate Risk Management Policy and Procedure'. A suitable pro-forma for recording the identified hazard and assessment is shown in Appendix C.

4. It is the preferred approach that the Designer develops strategies to mitigate those activities with high risk and include them in the Report. The residual risk must then be rated in accordance with MRWA Risk-Web matrix in the 'Corporate Risk Management Policy and Procedure'. Refer to the extract in Appendix D.

5. The residual risk must then be compared to what is considered acceptable by MRWA that is, Low or Medium.

6. The Designer must then document the risk in the Report. The Design Report must be signed by the Designer and endorsed by a Senior Designer before issue.

# APPENDIX B – EXAMPLES OF DESIGN RISKS

(Note that these are illustrative examples only and in every instance the hazard, likelihood and consequence must be assessed on its merits). Assessment of risk rating before and after Design may vary depending on specific site circumstances.

| Construction Activity | Designer Identified Hazard | Likelihood Level | Consequence Level | Risk Rating | Action Designer has taken to Reduce Risk | Residual Risk Rating | Residual Risk Acceptable? |
|---|---|---|---|---|---|---|---|
| Bridge Launching | • Uncontrolled movement of bridge during launching<br>• Locations of high load and stress | 1 | 4 | Medium | Designer ensures details of all design assumptions are included on the Drawings and Specification. | Low | Yes |
| Working over or near a Railway | Conflict between site staff/plant with<br>• trains and<br>• overhead electrified cables | 1 | 4 | Medium | Designer liaises with the Rail Authority at the design stage and identifies a process enabling the successful completion of works. Specifications require Rail Authority personnel to be present on site during critical stages of construction. | Low | Yes |
| Structural Alterations requiring Temporary Supports | Instability and possible collapse | 2 | 4 | High | Designer to provide the loads required for propping/temporary works | Medium | Yes |
| Lifting of large precast concrete members | • Instability during lifting<br>• Inadequate capacity in lifting points<br>• Access for cranes, capacity for lift<br>• Safe landing and location and fixing of precast member | 2 | 4 | High | Designer to provide lifting loads, position of lifting points, anchorage requirements for lifters. Also, seek advice from specialists if necessary. | Medium | Yes |

# APPENDIX C - DESIGN REPORT PRO-FORMA

| Construction Activity | Designer Identified Hazard | Likelihood Level | Consequence Level | Assessment of Risk | Action Designer has taken to Reduce Risk | Residual Risk Rating | Residual Risk Acceptable?<br><br>***Sign off*** |
|---|---|---|---|---|---|---|---|
| | | | | | | | |
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# APPENDIX D - MAIN ROADS RISK REFERENCE TABLES

## QUALITATIVE MEASURES OF LIKELIHOOD

| LEVEL | DESCRIPTOR | FREQUENCY |
|---|---|---|
| 1 | Rare | Less than once in 10 years |
| 2 | Unlikely | At least once in 10 years |
| 3 | Moderate | At least once in 3 years |
| 4 | Likely | At least once per 1 year |
| 5 | Almost certain | More than once per year |

## QUALITATIVE MEASURES OF CONSEQUENCE

| LEVEL | RANK | INJURIES | FINANCIAL LOSS | INTERRUPTION TO SERVICES | REPUTATION & IMAGE | PERFORMANCE | ENVIRONMENTAL |
|---|---|---|---|---|---|---|---|
| 1 | Insignificant | No injuries. | Less than $25,000 | Less than 1 hour | Unsubstantiated, low impact, low profile or no news item. | Up to 5% Variation in KPI or objective. | No lasting effect of significance.<br>Very short term impact (< 6 months). |
| 2 | Minor | First aid treatment. | $25,000 to $100,000 | 1 hour to 4 hours | Substantiated, low impact, low news profile. | 5% to 10% Variation in KPI or objective. | Minor localised impacts.<br>Short term impact (6 months - 2 years). |
| 3 | Moderate | Medical treatment required. | $100,000 to $500,000 | 4 hours to 24 hours | Substantiated, public embarrassment, moderate impact, moderate news profile. | 10% to 25% Variation in KPI or objective. | Localised - local significance.<br>Medium term impact (2 - 5 years). |
| 4 | Major | Death or extensive injuries. | $500,000 to $ 5 million | 24 hours to 1 week | Substantiated, public embarrassment, high impact, high news profile, Third Party actions. | 25% to 50% Variation in KPI or objective. | Severe and of moderate size.<br>Long term impact (5 - 20 years). |
| 5 | Catastrophic | Multiple deaths or severe permanent disablements. | More than $5 million | More than 1 week | Substantiated, public embarrassment, very high multiple impacts, high widespread multiple news profile, Third Party actions. | More than 50% Variation in KPI or objective. | Severe and extensive.<br>Permanent or very long term (> 20 years).<br><br>* Evaluate in terms of the scale and/or degree of impact. |

**RISK-WEB RISK RATING CHART**