LOAD RATING AND REFURBISHMENT DESIGN MANUAL

FOR EXISTING TIMBER BRIDGES

KALGAN RIVER (LOWER) – NO 4332

Structures Engineering

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AUTHORISATION

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

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REVISION STATUS

Clause No.	Revision No.	Revision Date	Revision Description
1.4.1	3	06/08/2018	Tables revised.
Preamble, 1.2, 1.4	2	22/06/2018	Introduction and Methodology amended for introduction of AS5100-2017 Bridge Series. Guidelines on members to be checked for stringers, piles and halfcap bearings amended.
General	1	09/01/2009	Major revisions throughout the document, Appendix C Added.

Custodian Endorsement

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Structures Design & Standards Engineer
Date: 06/08/2018

PREAMBLE

This Manual has been written to detail the process and methodology Main Roads Western Australia (MRWA) has adopted in the load rating and refurbishment design of timber bridges. It contains information on allowable stresses and modification factors for the typical Western Australian timber used in bridge construction as well as worked examples.

In preparing this Manual, MRWA has not adopted the current ultimate limit state design as per Bridge Design Code AS5100-2017 series, but has instead based it on the previous working stress code (AS 1720.1-1988). Reasons for this are provided within the accompanying text.

The Manual also provides an introduction to the design of refurbishment works to existing timber bridges. This is a subject not explicitly covered by text books, especially the specific repair/strengthening details which have been developed by MRWA.

1 LOAD RATING OF EXISTING TIMBER BRIDGES

1.1 Introduction

This Manual shall be used to determine the load capacity of existing timber bridges in Western Australia (WA), and to determine the load limit posting requirements for bridges with deficient load capacity.

Load ratings are carried out on bridges identified following a detailed inspection as possibly having insufficient capacity for the designated loads. If the load rating 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 aim of load rating analysis is to determine the theoretical capacity of a bridge by calculating what proportion of the T44 design vehicle, AS 5100 Bridge Design (CODE) M1600 design vehicle, and various other rating vehicles it can carry.

It is important to note that designing a bridge is different to load rating a bridge, where, in general, only **deficient** timber elements are investigated. As long as the structural configuration of the bridge, such as stringer diameters, spacing and span length are "standard" then it is acceptable to assume that elements having sufficient remaining solid timber are structurally adequate to carry the rating vehicles. A general arrangement of a typical, "standard" timber bridge is shown in Appendix A.

1.2 Methodology and General Approach

The methodology of load rating timber bridges is based on standard engineering principles and structural models, the CODE, the Working Stress Design Guidelines attached in Appendix B and computer modelling of spans, piers and abutments using the Timber Bridge Analysis and Repair (TIMBAR) suite of programs. The method of analysis is a **working stress** method, rather than limit state, and working or service loads are used in conjunction with the allowable stresses given in Appendix B.

The rating of timber bridges is based on the requirements of the working stress Timber Structures Code AS 1720.1-1988. The current ultimate limit state Bridge Design Code AS 5100-2017 series was not adopted as it appears to give overly conservative results compared to standards that have been used successfully by MRWA for many years.

MRWA has undertaken parallel assessments using both codes and then reviewed the varying results. Although the ultimate limit state code was intended to be a direct or 'soft' conversion from the working stress code, this proved not to be the case with respect to timber bridges. The two main factors causing the variance were found to be:

- AS 5100.9, Clause D1.3 states the Dynamic Load Allowance (DLA) shall be 0.25. This is lower than the value of 0.3 used in working state design; and
- Some of the capacity reduction factors adopted in the bridge code are conservative when considered in conjunction with the ultimate load factors adopted.

Timber bridges are simply supported structures, and the superstructure can therefore be analysed one span at a time. Timber bridges with concrete overlays are also analysed as simply supported, as the concrete overlay is designed as non-continuous and is designed to crack over the piers. Any continuity which may result from corbel to stringer interaction is ignored. Obviously, when assessing substructure elements the load effects from adjacent spans are included.

The rating vehicles comprise various axle groups, which are used unfactored to determine working load bending moments, shear forces and/or axial forces in the various structural members, including allowance for the Dynamic Load Allowance (DLA). Stresses from these load effects are calculated and compared to the allowable design stresses derived from Appendix B.

Refer also to the TIMBAR Procedure Manual.

1.3 Modelling of Concrete Overlays

Due to uncertainties in long-term composite action between overlays and timber decks, concrete overlays are not considered to contribute to strength or stiffness in the longitudinal direction, and are ignored in strength calculations for the timber stringers. However, they improve the transverse distribution of load, and are therefore modelled in the transverse direction in the grillage model. Concrete overlays are considered structural when assessing the local effects of the transverse timber decking.

The modelling of concrete overlays to timber bridges shall be based on the following criteria:

- Transverse members shall be concrete of the minimum structural depth of overlay.
- The use of full uncracked section properties for calculating the transverse member properties is considered to more accurately model what is actually happening in-service.
- Cracked section analysis, although strictly theoretically correct, tends to produce somewhat conservative results for bridges with borderline standard capacities.
- The use of concrete overlay properties for the transverse members models the increased transverse load distribution that the overlay provides.
- The concrete overlay is assumed to be non-composite in both the longitudinal and transverse directions.
- Timber decking is considered as dead load only (the effect of using the additive stiffness of the concrete plus the timber provides only a marginal increase in effective stiffness, and is not warranted).
- All of the overlay concrete over and above the minimum structural thickness is considered as non-structural and is therefore contributing to loading only as an additional dead load, applied as a trapezoidal patch load.

1.4 Members To Be Checked

When load rating timber bridges, generally the critical elements to be checked are stringers and piles identified as deficient in the detailed inspection report. However sometimes timber decking may warrant load rating if it is in a deteriorated condition or it is thinner than normal or if there is no road pavement present to distribute loads between planks.

Similarly sometimes timber sill beams, bedlogs or halfcaps are found to be in a deteriorated condition or are smaller than normal and may warrant load rating.

1.4.1 Stringers and Piles

As a guide, if the timber stringers have the minimum diameter as noted in the tables below, the stringers are considered structurally adequate, and normally would **not** require investigation for load rating:

Minimum Stringer Diameter - Main Roads Bridges – Jarrah – End Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<150mm solid timber section)	Type 2 Defect (<120mm solid timber section)
L=6.4m	340mm	400mm	425mm	350mm	400mm
L=6.1m	340mm	390mm	425mm	350mm	400mm
L=5.8m	340mm	390mm	425mm	350mm	400mm

Minimum Stringer Diameter - Main Roads Bridges – Jarrah– Midspan Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<150mm solid timber section)	Type 2 Defect (<120mm solid timber section)
L=6.4m	400mm	410mm	425mm	460mm	550mm
L=6.1m	385mm	390mm	420mm	430mm	500mm
L=5.8m	375mm	380mm	420mm	400mm	475mm

Minimum Stringer Diameter – Local Government Authority (LGA) Bridges – Jarrah– End Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<150mm solid timber section)	Type 2 Defect (<120mm solid timber section)
L=6.4m	310mm	375mm	400mm	330mm	350mm
L=6.1m	310mm	370mm	400mm	330mm	350mm
L=5.8m	310mm	370mm	400mm	330mm	350mm

Minimum Stringer Diameter - Local Government Authority (LGA) Bridges – Jarrah– Midspan Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<150mm solid timber section)	Type 2 Defect (<120mm solid timber section)
L=6.4m	380mm	390mm	410mm	415mm	520mm
L=6.1m	370mm	380mm	400mm	380mm	460mm
L=5.8m	360mm	370mm	390mm	370mm	430mm

Minimum Stringer Diameter - Main Roads Bridges – Wandoo – End Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<120mm solid timber section)	Type 2 Defect (<80mm solid timber section)
L=6.4m	300mm	350mm	370mm	340mm	400mm
L=6.1m	300mm	350mm	370mm	340mm	400mm
L=5.8m	300mm	350mm	370mm	340mm	400mm

Minimum Stringer Diameter - Main Roads Bridges – Wandoo – Midspan Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<120mm solid timber section)	Type 2 Defect (<80mm solid timber section)
L=6.4m	340mm	355mm	360mm	400mm	450mm
L=6.1m	320mm	340mm	340mm	375mm	435mm
L=5.8m	310mm	340mm	340mm	350mm	420mm

Minimum Stringer Diameter - Local Government Authority (LGA) Bridges – Wandoo – End Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<120mm solid timber section)	Type 2 Defect (<80mm solid timber section)
L=6.4m	280mm	320mm	340mm	310mm	370mm
L=6.1m	280mm	320mm	340mm	310mm	370mm
L=5.8m	280mm	320mm	340mm	310mm	370mm

Minimum Stringer Diameter - Local Government Authority (LGA) Bridges – Wandoo – Midspan Drillings

Clear Span (m)	Solid Stringer	Type 1 Defect (200mm diameter of rot or pipe)	Type 1 Defect (250mm diameter of rot or pipe)	Type 2 Defect (<120mm solid timber section)	Type 2 Defect (<80mm solid timber section)
L=6.4m	320mm	335mm	345mm	360mm	425mm
L=6.1m	300mm	315mm	330mm	340mm	410mm
L=5.8m	290mm	315mm	320mm	330mm	390mm

As a guide, the following remaining solid timber sections for piles are considered structurally adequate, and normally would **not** require investigation for load rating:

Jarrah Piles
100 mm solid annulus, with a minimum outside diameter 300 mm for piles with a maximum free height of 5.0 m.

Wandoo Stringers and Piles
80 mm solid annulus, with a minimum outside diameter of 300 mm for piles and a maximum free height of 5.0 m.

Bridges where the detailed inspection report shows the amount of solid timber remaining is less than these values, or diameter/lengths are outside the above criteria, shall be load rated as described in this procedure.

1.4.2 Halfcaps

Halfcaps may also need checking, but only if the report specifically states they are cracked, bowed, deteriorated or smaller than normal.

The following analysis method shall be adopted for the load rating of halfcaps:

- Assess which halfcap span and stringer combination is likely to cause the largest halfcap shears and/or bending.
- Apply vehicle loads to TIMBAR model, with wheels specifically located directly over the stringer/s identified above, by applying false kerbs in the model to restrict lateral vehicle movement – this will give the maximum shears and bending for one specific load case.
- The live load reactions at the pier support shall be distributed to the halfcaps 1/3:2/3 at each stringer support. The strength sharing factor, k_9, is not used for halfcaps.
- Shear Analysis - Vertical loads from stringers with centrelines within D/4 (where D = halfcap depth) from the face of the nearest pile are considered to be resisted by direct bearing, and can be ignored. Vertical loads from stringers with centrelines more than 5D/4 from the face of the nearest pile are considered to be resisted by shear in the halfcap, and the full load is taken for analysis. Vertical loads within the range D/4 and 5D/4 are considered to be resisted by a combination of direct bearing and halfcap shear, and can be reduced in proportion to the distance from the face of the pile, with zero shear at D/4 to 100% shear at 5D/4.
- Bending Analysis - Vertical loads from stringers with centrelines within D/4 (where D = halfcap depth) from the face of the nearest pile are considered to be resisted by direct bearing, and can be ignored. Vertical loads from stringers with centrelines more than D/4 from the face of the nearest pile are considered to be resisted by bending in the halfcap, and the full load is taken for analysis.
- Taking the position of the stringers into account for reduced loads and ignoring direct bearing as above, analyse halfcaps for shear and bending as beams supported at pile centrelines.

Refer to Appendix C for a worked example of the required halfcap analysis.

1.4.3 Deck Planks

Deck planks require pavement cover to help spread tyre loads and need to be checked where there is low cover or no cover and/or deterioration of the timber. As a guide, the minimum requirement for exposed timber decking is: -

Internal Decking Span	Rated Capacity
Up to 750 mm	>100% T44
Up to 875 mm	>100% VSR

Notes:

- Assuming 125 mm thick exposed jarrah decking with no pavement cover.
- Span lengths are between centrelines of stringers. Up to 1100 mm spans are common.
- VSR = 16.5t tandem axle, 20t triple or quad axle with tyre widths at 1.8m centre to centre.

Timber decking with spans in excess of these rule of thumb allowable spans should be checked to determine whether they have adequate capacity in shear and bending to resist the design loads.

Similarly bridges with timber deck thicknesses and/or effective thicknesses (due to deterioration) less than 125 mm will have lower allowable spans and should be assessed for effective load carrying capacity using the TIMBAR Deck Plank Workbook and hand calculations.

Where deck planks require rating, they shall be analysed as continuous beams over the stringers with span lengths taken between stringer centrelines, except where decking is known to be discontinuous.

Tyre loading shall be applied as a patch load based on the tyre contact areas given in the CODE. Patch loads may be distributed through any pavement fill material at 45 degrees.

Refer also to the TIMBAR Procedure Manual.

1.4.4 Halfcap Bearing onto Piles

Halfcaps with a 70mm bearing may not be theoretically suitable for bridges that have a reinforced concrete overlay or have small pile diameters. All halfcap/piles that are showing signs of distress shall be recorded and referred for further investigation.

Halfcaps which have less than the minimum width of bearing, 70mm for timber halfcaps or 50mm for steel halfcaps, onto timber piles, with a minimum diameter of 340mm, shall be checked for adequacy using the spreadsheet included in the TIMBAR suite of programs.

In order to assess the dead and live loads acting at the pile in question, the pier and the adjacent spans supported by the pier are required to be modelled and analysed using TIMBAR. The relevant pile reactions are then inserted into the halfcap bearing spreadsheet to determine the adequacy of the halfcap bearing area provided and/or whether additional bearing support is required.

Refer also to the TIMBAR Procedure Manual.

1.5 Stress Grades

Tables B2.1 and B2.2 in Appendix B contain strength classification and design densities for common groups of timbers and species. To provide consistency and based on practical experience, MRWA has selected the appropriate stress grades for the commonly used bridge timbers. Unless otherwise specified or assessed, the stress grades to be used for the various timbers used in the different structural elements are shown in the Table below:

ELEMENT	JARRAH	KARRI	WANDOO	MARRI
Stringers (round)	F17	N/A	F27	F22
Stringers (sawn) *	F14	N/A	F17	F17
Piles	F17	N/A	F27	N/A
Halfcaps (sawn) *	F14	F22	F17	N/A
Waling (sawn)	F7	F8	F11	N/A
Bracing (sawn)	F7	F8	F11	N/A
Decking (sawn)	F7	F8	F11	N/A
Bedlogs (round)	F17	N/A	F27	N/A
Bedlogs (sawn)	F14	N/A	F17	F17
Corbels (round)	F17	N/A	F27	N/A
Corbels (sawn) *	F14	N/A	F17	N/A
Sill beams (round)	F17	N/A	F27	N/A
Sill beams (sawn) *	F14	N/A	F17	N/A
Other	F7	F8	F11	N/A

* Sawn stringers, halfcaps, corbels and sill beams above have an assumed default grading of Structural No. 3 seasoned timber, which should be typically adopted unless a visual assessment can demonstrate that they are of a lower or higher grade than this.

Higher strength grades of timber than tabulated above shall only be used if visual inspection and grading has confirmed that the timber is of a stress grade higher than Structural No. 3 for seasoned timber. Refer to Table B2.2 of Appendix B, for stress grades of timber. Note that any plans to adopt higher stress grades shall be supported by evidence to show that this is the case, such as confirmation that the timber defects found on site differ to those tabulated in AS2082-2000 'Visually Stress-Graded Hardwood'.

Typically all timber elements with defects such as sloping grain or large gum veins in areas of maximum bending or shear stress should have their basic allowable stresses as tabulated above suitably reduced.

The average shear stress for stringer ends and sawn timber elements such as halfcaps is to be calculated by P/A_s (where $A_s = \frac{2}{3}A$ and A is the area of sound timber).

For Bedlogs, Corbels, Sill beams or any other timber elements subjected to localised bending perpendicular to the grain, as Appendix B provides no guidance on allowable stresses for bending in this direction, use a basic allowable stress perpendicular to the grain of 0.6 MPa for Jarrah and 0.9 MPa for Wandoo.

For round stringers with a spiral grain that is causing some opening up of the surface timber reducing interlock between grains or that is causing splits to occur at sawn discontinuities, adopt the allowable stress grades for sawn timber in Table B2.2 of Appendix B.

Some bridges in the south west of Western Australia have been built using Marri stringers. It has been noted that some Marri stringers on these bridges have failed and it is our current belief, at this stage, that Marri stringers may not have a friable condition as do Jarrah and Wandoo stringers, but deteriorate directly to a rot state with no intermediate friable phase.

1.5.1 Load Duration Factor (k_1)

Typical values for the Load Duration Factor, k_1, taken from Appendix B (Clause B2.5.1.1), which are to be used in load rating analysis, are as follows:

- for timber bridges on main roads k_1 = 1.40 (5 month duration of peak load)
- for timber bridges on local roads k_1 = 1.65 (5 day duration of peak load)

The higher value for the local roads reflects the lower traffic volumes, and hence shorter duration of load, and conversely, the lower value for main roads reflects the higher and heavier traffic loading which would produce a longer overall duration of loading for the bridge.

Local roads with traffic volumes exceeding 500 vehicles per day should be treated as main roads when determining the appropriate k_1 factor.

1.5.2 Material and Application Factor (k_2)

Typical values for the Material and Application Factor, k_2, are shown in Appendix B in Table B2.11.

Due to the inherent redundancies built into timber bridge construction it has been assessed that the k_2 factor to use as a default value for load rating of timber bridges is 1.0 for all structural elements.

1.5.3 Parallel Support Factor (k_8)

Typical values for the Parallel Support Factor, k_8, are shown in Appendix B in Table B2.9.

For typical timber bridges the only parallel support members provided to which this factor could apply are halfcaps. For these elements it has been assessed that the k_8 value to be used as a default for load rating is 1.0, to adequately allow for the fact that halfcaps are in effect assessed individually when calculating the maximum load in any halfcap.

1.6 Deteriorated Timber

When rating timber elements with some remaining solid timber, all areas of timber identified as ‘rot’ or ‘friable’ shall be ignored when calculating section properties for both analysis and calculation of resulting stresses.

For the rare case of timber elements with no remaining solid timber (i.e. the drillings indicate all rot or all friable), a conservative approach is taken, as accurate modelling is not possible since the Young’s Modulus for rotten and friable timber is not known. Upper and lower bound cases are studied, i.e. ignoring the stringer, (to give the maximum load in adjacent stringers) and assuming it is solid (to give maximum load in the defective stringer). When calculating the resulting stress, the following reduced allowable stresses shall be used:

Compression	Rot	= 15% of Allowable F'_c
	Friable	= 85% of Allowable F'_c
Tension	Rot	= 10% of Allowable F'_t
	Friable	= 70% of Allowable F'_t

Shear	-	**for stringers & piles;** as per Tension above;
	-	**for halfcaps;** as per Tension as above, plus a 50% overstress is allowed where the distance between the centreline of the corbel/stringer and the edge of the pile is less than the depth of the halfcap.

The above values are estimates only. Further laboratory testing of different timber elements in various conditions is required to confirm the correctness of these estimated strength values for friable and rotten timber.

However, initial test results indicate that the above values are probably a lower bound answer, and hence conservative. These values have been adopted and included as the default settings in the TIMBAR programme which is generally used for analysis when carrying out load ratings and refurbishment designs as described in the following sections.

At this stage, it would therefore appear to be appropriate to ignore any friable timber reported in Marri stringers and treat as rot accordingly.

1.7 Geometry and Dead Load Analysis

Drawings for old timber bridges are generally unavailable. A General Arrangement of a typical timber bridge is attached at Appendix A.

For the geometry of the superstructure, and dimensions of the various elements, information can be obtained from Detailed Timber Bridge Inspection Report and the Bridge Inventory database.

For the calculation of dead load all timber structural elements, no matter what their condition, shall be taken as solid with the density of sound timber. The density of all WA hardwood timber species shall be taken as 11 kN/m^3.

Gravel and/or an bituminous surfacing are the typical pavement material on a bridge deck. The superimposed dead load effects of both are calculated assuming a density of 22 kN/m^3. The depth of the pavement material may be estimated from the depths at the four corners of the bridge deck, recorded in the detailed timber bridge inspection, with appropriate allowance made for cross falls as shown on the drawings.

For the edge stringers, an additional 1 kN/m shall be applied to represent the weight of the guardrails plus typical kerb (0.3m wide by 0.1m deep). Large concrete kerbs and services carried are additional and should be added separately. Refer to TIMBAR procedure Manual.

Appropriate Patch Loads shall be input into TIMBAR to accurately reflect the actual dead loads and superimposed dead loads acting on the structure, deducting the weight of dead loads already input for the transverse elements (based on density and thickness).

Where the patch loads vary across the deck, trapezoidal patches should be input to accurately reflect the changes in thickness to suit. In general it is good practice to sketch a cross section of the deck showing the location and magnitude of all Patch Loads proposed to be adopted in the model, to allow for ease of checking and review of assumptions.

1.8 Live Load Analysis

A grillage analysis of the deck is usually carried out using TIMBAR to determine the live load distribution to each stringer. This enables calculation of moments and shears in the stringers and loads to be applied to the structure.

The substructure is usually analysed as a plane frame using TIMBAR, with the piles, both pier and abutment, considered as pinned at the top and fixed at a depth into the ground. Abutment piles also have to carry the horizontal loads from the road fill.

As the transverse distribution of timber bridge decks (with or without concrete overlays) is not very good, one vehicle directly above or as close as possible to the timber element in the superstructure under investigation is usually sufficient to determine the worst load case. Similarly, for the substructure, where the lack of stiffness of the timber halfcaps to distribute the loads from the stringers to the piles means one vehicle with its wheels straddling a pile will generally give the worst load case. The exception to this is where the bridge has steel halfcaps which can distribute the stringer loads more effectively. In this case two vehicles side by side may need to be investigated to determine the worst load case for the substructure.

The TIMBAR software suite has been specifically developed by MRWA, and is based on the ACES Bridge Analysis System. This program allows easy input of the timber stringer properties based on the inspection data and generates a grillage model complete with relevant longitudinal and transverse section properties. Rating vehicles are then automatically moved along and across the bridge span to generate the worst effects. Rated capacities are tabulated for each vehicle for bending and shear actions and a summary of the controlling ratings presented. Where possible, the use of TIMBAR is the preferred analysis tool for load ratings and refurbishment designs of timber bridges, as this will ensure consistency and compatibility of results.

Two sets of span lengths are entered into TIMBAR for the span analysis:

1. The centre to centre spacing between pier centrelines and
2. The clear distance between the ends of corbels (clear span length),

both of which are available from the Detailed Inspection Report for each span.

Effective span lengths for stringers, to take into account corbel action (where present), are automatically calculated by TIMBAR, being taken as half way between pier centrelines and the ends of corbels.

TIMBAR uses span lengths for pier, abutment and halfcap analysis based on the measured centre to centre distances between piers and/or abutments as measured in the latest Detailed Timber Bridge Inspection Report.

1.9 Load Rating Vehicles

Load rating vehicles for timber bridges are shown in the TIMBAR Procedure Manual.

Except for the T44 and M1600 loading, only one of the various rating vehicles is assessed to be on the bridge at any one time with no coincident loading. For T44 and M1600, multiple lane loading must be used, as required by the CODE. Unless otherwise specified, the M1600 shall be checked and recorded, but shall not govern the rating or refurbishment design of timber bridges.

In general, most timber bridges are simply supported with spans typically between 6 and 7 metres. Therefore, only one axle group of the vehicle will fit on any one span.

For structures with larger spans, and/or for the rating of substructure members, the load effects of the adjacent axle group/s must be included. The axle weights of the prime mover are controlled by law and are assumed to be constant for purposes of the assessment as 6 tonne on the steer axle and 18 tonne on the tandem drive. The Prime mover is not however modelled in TIMBAR.

The rating values for vehicles, the percentage rating, and the value of weight for the other vehicles are used in the assessment of heavy load movements, and to determine a load posting limit where required.

1.10 Required Rating Information

The information required from the load rating exercise must identify the bending capacity mid span and the shear capacity at both ends of each stringer, bending and shear for piles, halfcaps and decking planks, for each of the vehicles or axle groups listed in the TIMBAR Procedure Manual.

The report must contain:

- bridge number
- span and pier numbers
- stringer and pile numbers
- stringer and pile section properties
- stringer and pile section capacities for bending, shear and axial loads as appropriate
- halfcap, sillbeam, corbel and decking properties
- halfcap, sillbeam, corbel and decking section capacities
- rating determination for each of the rating vehicles
- a summary table of the controlling rating for each vehicle and critical member

The report shall detail the assumptions used in the analysis and the program(s) used to model the timber bridge. The critical ratings for each structural component are to be listed, identifying the limiting element for each rating vehicle. TIMBAR output, as described above, must also accompany this load rating report for timber bridges. An example of the preferred format is given in the TIMBAR Procedure Manual.

2 REFURBISHMENT DESIGN OF EXISTING TIMBER BRIDGES

2.1 General Refurbishment Methods

There are approximately 1330 timber bridges in Western Australia, of which about 350 are on Highways and Main Roads. Many of these bridges are deteriorating due to weathering and fungal attack, and refurbishment methods have been developed to preserve and where necessary, strengthen these structures.

The main method of prolonging the life of a timber bridge is by placing a reinforced concrete overlay onto the deck. The main effect of the concrete overlay is to provide a weatherproof deck to protect the timber decking and stringers, and it also increases stiffness and reduces vibration. As the roadway width of timber bridges is frequently below current standards, bridges are often widened at the same time, and this is incorporated into the concrete overlay design. The widening also provides a “veranda” effect for the outside timber stringers, the most exposed bridge members, thus reducing weathering of the timber. Under these conditions, the life of the bridge timbers is prolonged significantly.

Early concrete overlays were designed as thin, flexible running surfaces providing a weatherproof cover to the timber deck. Consequently many of the overlays carried out in the early 1970s are now failing, having badly cracked due to their flexibility under traffic loading, and are leaking water. Concrete overlays designed today are thicker and contain more steel reinforcement than the earlier overlays. With increasing guardrail requirements, another important consideration for the concrete overlay is to provide adequate connection for a guardrail appropriate to the current standards.

Load tests of bridges with concrete overlays indicate that partial composite action may be developed (although this effect is not included in the design).

Repairs to the superstructure (i.e. stringers) and substructure (i.e. piles, halfcaps, wingwalls, etc) are also a major consideration when refurbishing a bridge. It is preferable that this work is carried out together with the concrete overlay.

2.2 Standard Details

2.2.1 General

As nearly all timber bridges are of identical construction (refer Appendix A for typical timber bridge details) most repair and concrete overlay details have been standardised. Structures Engineering has prepared a manual of standard details called the “Structures Engineering Practice Notes” (Practice Notes).

These details have been enhanced and modified to achieve the most economical design utilising common materials and available construction expertise, whilst maintaining acceptable design standards. This continual improvement of the standards is now at the stage where they are issued as a controlled document, to allow updates and modifications to be sent to each document holder, enabling all users to have the most up to date and recent design details referred to as, the “Structures Engineering Practice Notes”. This manual should be referenced for more detailed information on the standard repair and concrete overlay details discussed below.

The following sections discuss some of the more important and common repairs such as pile repairs, new steel stringers and halfcaps, and deck widening by cantilevering from the existing structure.

It should be noted that the Standard Details apply to "normal" situations, and standard or typical timber bridges. Each Engineer applying these details should check that the detail being used is applicable for each particular bridge and loading.

2.2.2 Pile Repairs and Replacements

It is quite common to have to repair or replace timber piles on bridges because these elements are in contact with the ground which is a very high risk area for rot and decay to occur.

Where a timber pile has deteriorated to the extent that it is under capacity for the design loads or it has less than the mandatory accepted solid timber remaining (as prescribed in Section 1.4.1 of this document), then it should be considered for repair or replacement taking into account historical deterioration rates. A repaired pile typically incorporates a new steel column section joined by a concrete collar onto the remaining solid timber pile below the ground, making use of the original timber foundation.

Where the length of the original timber pile above ground level exceeds 2.5 m (and the timber is in good condition) the remaining timber section of a repaired pile is generally retained following the removal of the deteriorated section. Where the length is less than 2.5 m, then the steel column section shall extend up to the halfcaps.

Figures PN30-2103 to PN30-2133 of the Practice Notes detail typical timber pile repairs.

Axial load capacity is maintained by the steel column section, and moment capacity by the concrete pot. This type of repair has been carried out on a large number of timber bridges over many years and has proved very successful and economical. The halfcaps need to be adequately propped during the repair works, and also, where possible, all traffic directly above the pile being repaired should be diverted to the other side of the bridge, and a speed limit imposed.

This type of repair has been designed to be equivalent to the original timber pile, and is capable of safely carrying an axial load of 500 kN, and a moment of 150 kNm, based on a height of pile above ground not exceeding 3.5 m.

In cases where repairing the timber pile is considered inappropriate then the pile is generally replaced by driving a new steel pile alongside the deteriorated timber pile.

Splitting of timber piles below the halfcaps is very common, due to bearing and seating problems at the pile notch. Splits can also occur at other locations along the length of the pile. Where such splits occur, the piles are banded with steel to radially compress the pile and close the split, and to prevent the split expanding under load.

2.2.3 Cantilever Widening

During construction of a concrete overlay, additional bridge width can be obtained at relatively small cost by extending the cantilever on one or both sides of the bridge. Where the extent of widening is such that the capacity of the reinforced concrete overlay is insufficient by cantilevering, then new steel stringers are installed supported by extended halfcaps, typically new steel channel sections. This is a cost effective solution and is quite common. A more expensive alternative is driving new steel piles and installing new steel halfcaps. This is usually only required when the widening is substantial, or is on one side of the bridge only.

Figures PN30-2321 to 2324 and PN30-3204,3205 and 3207 of the Practice Notes show typical details of a cantilevered widening or bridge widening with new steel stringers and halfcaps.

The concrete slab widening is affected by the use of permanent formwork (typically Bondek) or in corrosive environments an off form concrete finish will need to be used. The deck spans transversely from the centreline of the existing outer timber stringer to the kerb, cantilevering past any new steel stringer as required, depending on the extent of widening. The maximum spacing between the existing outer timber stringer and the new steel stringer is generally the same as that used for timber stringers, which is about 1.0 to 1.2 m.

Cantilevering of the concrete beyond the last existing timber or new steel stringer is kept to a maximum of 500 to 600 mm, based on the capacity of the overlay in bending and shear.

2.2.4 Guardrails

The new concrete kerb details are shown in Practice Notes PN30-3202 to 3207, and allow the existing substandard timber post and rail system to be replaced with a new steel post and Thriebeam guardrail system. The kerb to post fixings allow damaged posts to be replaced with little or no damage to the concrete kerb, and therefore quick and economic repairs.

Top rails are generally not required for most bridges except for high structures (more than 4.0 m from deck to ground) with significant traffic volumes and where the likelihood of pedestrian or cycle traffic is also high.

Figures PN30-4101 to 4105 and PN30-4108 of the Practice Notes show typical Thriebeam and W-beam guardrail requirements.

2.2.5 Stringer Strengthening/Replacement

Where a load rating analysis indicates that an existing stringer is under capacity or the stringer has less than an acceptable minimum solid timber annulus remaining (any drillings under 60 mm solid would be a typical acceptable minimum), then the stringer is either replaced with a steel beam or a steel beam is placed alongside the deteriorated stringer. Replacing the stringer is usually preferred.

The typical steel replacement used in stringer strengthening repairs is the 410UB54 (although the beam size will need to be checked for each strengthening design). This size beam is of a similar size to the existing timber stringers, has similar stiffness and generally provides adequate strength. As the stringer height at the corbel is typically around 350 mm, the depth of the new steel stringer is reduced at the corbel seating to fit into the available space.

All steelwork used in timber bridge refurbishments is hot dipped galvanised for corrosion protection. All welding is special purpose structural welds and requires a qualified and skilled welder. As such, site welding is minimised with many of the steel connection details bolted.

2.2.6 Halfcap Strengthening/Replacement

New steel halfcaps can be installed either beneath the existing timber halfcaps, (halfcap strengthening), or installed to completely replace the timber halfcaps (halfcap replacement). The condition of the existing halfcaps determines which of these details is used.

Where the existing timber halfcaps are in reasonable condition then the new steel halfcaps are generally installed beneath the existing timber halfcaps, with existing vertical corbel bolts recessed into the bottom of the timber halfcaps and the new steel halfcaps located immediately below the timber halfcaps, with minimal packing using galvanised steel packer plates welded to the steel halfcaps. The new steel halfcaps are bolted horizontally at each pile and both sets of halfcaps are bolted together vertically. The timber piles are notched to provide seating for the steel halfcaps, and may require supplementary steel plates or channel sections for this purpose if the piles are out of alignment and sufficient seating cannot be gained. Refer PN30-2328 of the Practice Notes for details.

Where the existing timber halfcaps are in poor condition then they are generally totally replaced by new steel halfcaps. The steel halfcaps are bolted horizontally to the pile (similar to strengthening) while angles are welded either side of the corbel/stringer and bolted horizontally to restrain it. For halfcap replacements, the corbels/stringers require propping and jacking to allow removal and installation of the halfcaps. Propping is obviously difficult for piers over permanent water. In these situations a constructability versus long term maintenance cost assessment must be undertaken to assess if the halfcap should be replaced or strengthened.

2.3 Design Procedures

MRWA has design procedures as outlined in Part 5 of the “Structures Engineering Management System” (SEMS), Document No. 3912/01, which shall be followed for internal use. All necessary steps, flow charts and Control Sheets shall be followed, completed, signed off and kept as hard copies on the relevant bridge file for future reference.

Consultants providing a service to MRWA will have their own design procedures, however there are specific control sheets that MRWA requires to be used. They are the Design Warrant (Form 3912/01/05/03), the Design Criteria Sheet, the Bridge Width Approval Form, the Design Control Sheet (Form 3912/01/05/01) which are available on request.

2.4 Typical Refurbishment Design Process

The design process adopted for a typical refurbishment design has been detailed below. Although primarily written for internal MRWA use, the process is largely generic and may be adopted for wider use.

2.4.1 Data Gathering and Recording

- Receive a verbal or written brief and timeframe on the extent of work programmed for the particular bridge.
- Retrieve the current bridge maintenance file and old 51 series files, obtain any existing drawings and obtain a copy of the latest Detailed Timber Bridge Inspection Report (if not already on file).
- Arrange for a new detailed inspection and report to be undertaken if the latest one on file is more than 3 years old.
- Copy standard blank control sheets as above from Part 5 of SEMS and place them on the maintenance file.
- Obtain traffic volume information (AADT) for the bridge site for completion of the Bridge Width Approval Form and to allow determination on any widening required to the existing structure as part of the refurbishment design.
- Obtain guidance from Engineer Bridge Loading (EBL) on the appropriate Design Vehicle Loads to be adopted in the design and include in the Design Criteria Sheet.
- Complete the Design Warrant, Design Criteria Sheet and obtain approvals and signatures.

- Continuously update progress on the Design Control Sheet, signing and obtaining signatures as the design progresses through the various stages and Control Points listed, attaching the Control Sheet with corner removed to the front of the bridge maintenance file (where it is to remain until the design process is completed).

2.4.2 Preliminary Design and Analysis

- From traffic data and existing bridge geometry review preferred minimum bridge width. If widening is found to be required ascertain whether the Region has allowed for this widening in their budget for the proposed works.

- If widening has not been allowed for but is found to be required to conform to the preferred design standards, liaise through your supervisor with the Region to either increase the scope of work to allow for widening or agree to accept no widening with a strategy for future allowance for widening or replacement of the structure. At this point the agreed scope for widening must be ratified by the Region in writing, together with any proposed future strategies for the bridge which have been agreed to.

- Commence preliminary design and analysis, load rating all elements listed on the Programme of Works for refurbishment and/or considered by inspection to be of questionable capacity or non-standard size and ascertain the current capacity of the bridge. Using the required new bridge geometry, calculate the capacity of the bridge in its current condition and with proposed loadings applied.

- From the above, determine inadequate elements that will require strengthening or replacement based on their current condition (inadequate due to widening or increase in loading proposed or deterioration in condition of existing timber elements), comparing also to the current capacity and adequacy.

- Prepare a full list of all elements proposed to be replaced or strengthened. Compare this list of elements with those allowed for in the Region’s Programme of Works for the bridge.

- Where timber elements (stringers, piles, decking, corbels, halfcaps, sheeting etc) currently have adequate capacity but are deteriorating and have only marginal reserve capacity for the design vehicle(s), list these elements separately for possible inclusion into the proposed scope of work. As a general principle, it is far more efficient to replace marginal but adequate elements at the same time as other inadequate elements are being replaced than to wait a few years and then have to program the bridge to have these other elements replaced because they have deteriorated further. Discuss these elements and the proposed strategy with your supervisor and include as an agenda item for the 15% Review Meeting with the Region’s Asset Manager Structures (AMS).

- Check whether a Whole of Life costing (WOL) has been carried out for the bridge. A whole of life costing will normally have been used by the AMS as the basis for determining what to include on their Programme of Works for the bridge and when to allow for the work to be carried out.

- If a WOL has been carried out for the bridge, check whether the work proposed to be carried out matches the assumptions used in the WOL. If not, another WOL may be recommended to be carried out by the Region to see whether the proposed refurbishment work is still the best course of action to be undertaken at the bridge.

- Compare the scope of repairs and refurbishment required from analysis (and due to likely further deterioration of timber elements) with that allowed for in the Region's budget for the bridge. Where the latest detailed inspection report shows significant further deterioration of structural elements or more widespread deterioration from the previous detailed inspection report upon which the Region's programmed works have been based, this may indicate that an increased scope of works for refurbishing the bridge should be allowed for.

- If preliminary analysis and assessments indicate that any change to the extent of work allowed for and budgeted for in the Region's Programme of Works is required, then the design engineer shall liaise, through his Supervisor, with the Region's AMS to discuss and agree on whether the scope and the budget can be increased. This can be done whilst on site with the Regional personnel (2.4.3) or can wait until the 15% Design Review Meeting (2.4.4).

- Budgeting and programming by the Region does not normally involve any detailed analysis but is based on engineering judgment, using the current Detailed Bridge Inspection Report, which could be up to 5 years out of date by the time the bridge is actually scheduled to be designed for refurbishment.

- For short, low bridges with headroom less than 1.0 metre, due to the difficulty of being able to adequately carry out detailed timber bridge inspections, it is preferred that these bridges be replaced with a new structure such as concrete bridge or culverts. If a replacement structure has not been allowed for in the Region's Programme of Works, this should be discussed with the Region's AMS – and possibly defer major repairs until condition warrants replacement.

2.4.3 Site Visit

- A site visit is standard practice if it is practical. Obtaining input from the Region (AMS and Project Manager Structures (PMS)) and Structures Engineering (Structures Delivery Standards Engineer (SDS)) on site is critical.

- Ascertain who the proposed draftsperson is going to be and arrange for him/her to also be present at the bridge site meeting.

- Obtain all necessary site measuring equipment and tools for the site visit, including protective footwear, safety vests, 8 and 30 metre measuring tapes, digital camera, clipboard, pig spike, cordless drill (if available), hammer, Wellington boots (if wet or boggy site).

- Visit the site and measure all relevant existing geometry to confirm or supplement the inspection report information and tabulated dimensions. Key items to measure include the existing pavement thicknesses (confirm report thicknesses), road approach alignments and gradients, whether the bridge is centred on the road, skew angles, whether widening is to be symmetric to best fit the road, vegetation, fire hazards, heights of existing barriers, noticeable cracks in pavement or concrete elements, timber visual condition. Check stringer and pile spacings, typical span lengths etc.

- Confirm on site whether the extent of work previously programmed such as banding, bolting, preventative and routine maintenance items is still accurate.

- Take photos of areas that are difficult to sketch or measure or that simply have not been photographed before. Seek the draftsperson's advice on photographs that would also be valuable for future reference when drafting.

- Record all existing services and other attachment details to enable removal and/or relocation.

- Measure existing pavement and shoulder widths, existing geometry of approaches and embankments, sufficient to complete the Bridge Width Approval Form and to allow design of approach barrier railing back in the office (i.e. with barrier splays, embankment lengths, slopes, vegetation and available space considered). Determine and agree the length of new barrier railing required off the bridge in all four corners.

- Discuss with the AMS and the SDS whilst on site the current understanding of the scope of work required at this point, taking into account what has been found from preliminary analysis done in the office, what has been seen and measured on site, what new items have come to light whilst on site etc. Compare additions and deletions when compared to the work items allowed for in the programmed work and agree with the AMS and SDS on any changes to the programmed work required.

2.4.4 15% Design Review

- Prepare 15% report on proposed scope and submit to the Region for approval.

- Once approval is received, arrange an internal 15% meeting.

- Record 15% Design Review Meeting minutes documenting any changes to scope or details suggested. Then, based on these changes, modify the design notes, sketches and proposed preliminary design items as required, and sign off the Control Sheet.

- If any significant changes to 15% scope or concepts have occurred in the 15% Design Review Meeting, advise the Region and obtain their approval and acceptance of proposed changes.

- If existing services are affected by proposed works then advise the Region's PMS and ensure he/she liaises with the relevant authorities to arrange to have the services moved (or temporarily disconnected and supported).

2.4.5 Detailed Design

- Commence detailed design based on the agreed 15% scope. This should involve load rating analysis of all affected spans and affected piers, halfcaps etc. for the new geometry and proposed new loads. Photocopy relevant standard details proposed to be used (and check capacity where required) and include as part of the Design.

- Prepare design sketches where standard details are not appropriate. Prepare a summary of the scope of work in line item form, summarising bolting and banding types and locations, routine, specific and preventative maintenance items, new stringers, replacement stringers, pile pots, halfcap strengthenings etc. Sketch a typical cross-section of the proposed new superstructure, plus any other sketches necessary to fully describe all work required.

- Where lane closures are required which will cause additional temporary increases in loading to be applied to certain elements, carry out analysis for these elements and prepare summaries of specific requirements to be adopted (such as temporary propping or stringer strengthenings or temporary traffic restrictions etc), including these in the design notes for the 85% Review to follow. Refer also requirements given in Section 2.5.

- Liaise with the Region to obtain all necessary clearances or feedback from relevant Authorities and Organisations listed on Form 3912/01/05/02 of part 5 of SEMS and, when completed, sign off all items required, attaching any relevant correspondence on the bridge maintenance file with folio numbers added to the form.

- Liaise directly, as required, with the Region and senior engineers for their input, advice or preferences on proposed methods of repair, where these proposed methods are non-standard or unusual, submitting preliminary sketches if necessary to explain the situation.

- Obtain guidance from senior engineers on approach barrier rail considerations and show required approach barrier geometry on sketches.

- Where non-standard bridge barriers are required, obtain guidance on appropriate alternative barriers, geometries and connection details and include in the design sketches.

- Put all of the above sketches, information and summary scope on the bridge maintenance file as a full record of your Design, deemed at this point to be 85% completed. Do not put your calculations on file. These should be filed separately in a suitably identified lever arch file and held within Structures Engineering for at least 7 years.

2.4.6 85% Design Review

- Submit 85% Design documents to the Region's AMS for review and comment.

- Once the Region's comments and approval have been received, place all correspondence received on file and call an 85% Review Meeting.

- Record 85% Review Meeting minutes and obtain appropriate approvals, or if approval is not received, carry out changes and further analysis and resubmit for 85% approval.

- If any significant changes result from the 85% Design Review meeting advise the Region for further review and acceptance of proposed changes.

- Make any necessary changes to the scope of work and design sketches that are required to the design as a result of the 85% Design Review meeting and place on the bridge maintenance file.

- Complete Control Sheet signatures for 85% review and commence final drafting.

2.4.7 Drafting

- Brief the draftsperson on the full scope of work required, issuing the final design notes, scope and sketches on the bridge maintenance file for reference and go through all items in detail.

- Attend to any drafting queries and, once drafting is complete, check through the drawings, marking up with any changes required in red. Sign off as checked.

- Issue the marked up set of drawings also to the Structural Information and Standards Manager (SISM) for review, mark up and sign off.

- Once the drawings have been changed as a result of engineering and drafting reviews and the back drafting is complete, submit for 100% Review, then sign (as designer) and obtain signatures on the drawings from Senior Design Engineer (SDE) (as verified) and Senior Engineer Structures (SES) (as approved).

- Arrange for drawings to be sent to the Region for signing. Liaise with SISM and the PMS for sizes and numbers of signed drawings required, as well as ascertaining whether electronic copies are required.

2.4.8 Specifications

- Review the standard specifications for Bridgeworks, in particular the 800 series of specifications and confirm that all proposed construction activities are adequately described by these specifications.

- Where a new or different type of repair/refurbishment activity proposed is not adequately described in these specifications, the design engineer shall liaise with the Region's PMS to prepare new descriptions of the work required (to be read in conjunction with details on drawings) and either include in the drawing notes, on the detail itself or prepare a special clause to be added to the specifications (to be reviewed and approved by the design supervisor).

2.4.9 Construction Support

- Provide construction support as required or as requested, including visits to the site to inspect the work or construction difficulties (only at the request of the Region).

- Answer any site queries during construction that arise, ensuring that all correspondence is confirmed in writing, is discussed and approved by the design supervisor prior to responding to the Region and a copy of all correspondence is kept on the Maintenance file for the bridge.

2.4.10 As-Constructed Drawings

- When As-Constructed Drawings for the bridge refurbishment have been submitted back to Structures Engineering from the Region for addition of as-constructed details to the drawings, liaise with SISM and arrange to review the As-Constructed Drawings to confirm that any changes are considered acceptable and have not compromised the design.

- Ensure any significant changes to the design, carried out on site with or without your knowledge, are fully acceptable to Structures Engineering, liaising with the design supervisor for guidance as required.

A flow chart for the Refurbishment and Bridge Strengthening Design Process is included in Appendix D.

2.5 Required Design Analysis

The steps described in **Section 2.4** are typical for most full refurbishment designs. The extent of analysis, detailing and design steps may be considerably reduced in some instances (i.e. minor strengthening).

The design analysis required for most refurbishment designs shall consist of rating the bridge (each span and pier and abutment as necessary) with deficient elements either replaced with new steel and/or concrete elements or strengthened with additional steel or concrete elements such that the required load capacity for each specified design vehicle is achieved. It is recommended that the designer checks at least one span, pier, abutment, halfcap, etc prior to finalising the design.

For a proposed bridge widening (usually in conjunction with a new Reinforced Concrete (RC) overlay), all spans should be load rated taking into account the proposed new geometry and overlay thicknesses. This ensures that there is a record of the widened structure's new capacity, for revision of and entry into the bridge inventory database as required. Similarly the existing substructure elements should also be load rated, or at least the lowest previously rated spans, piers, abutments and halfcaps should be load rated assuming the new bridge geometry and loads, to determine whether the substructure elements have now become the lowest load capacity elements.

All elements found to be deficient for the design loads specified in the Design Criteria Sheet shall be strengthened accordingly.

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

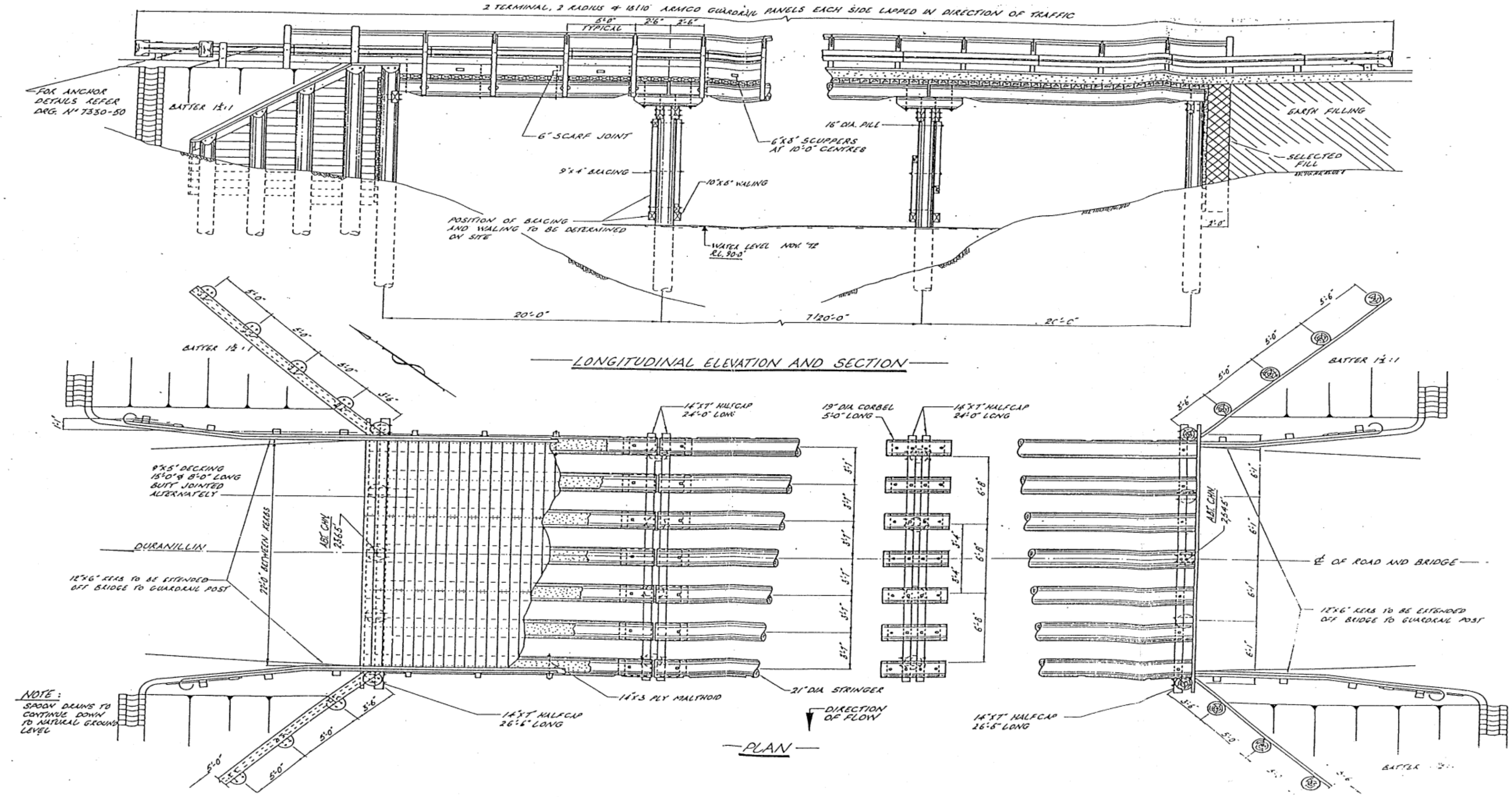
During construction of a refurbishment or strengthening design it is common for the road to remain at least partially open to traffic. The Designer must provide for this in the design by either propping various elements as required or designating traffic restrictions that are required.

The bridge must be analysed for each stage during construction (for example, a two stage RC Overlay construction), giving allowable traffic loads as outlined in the Practice Notes. This information is required for heavy load assessments during construction and shall be forwarded to the Engineer Bridge Loading (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.

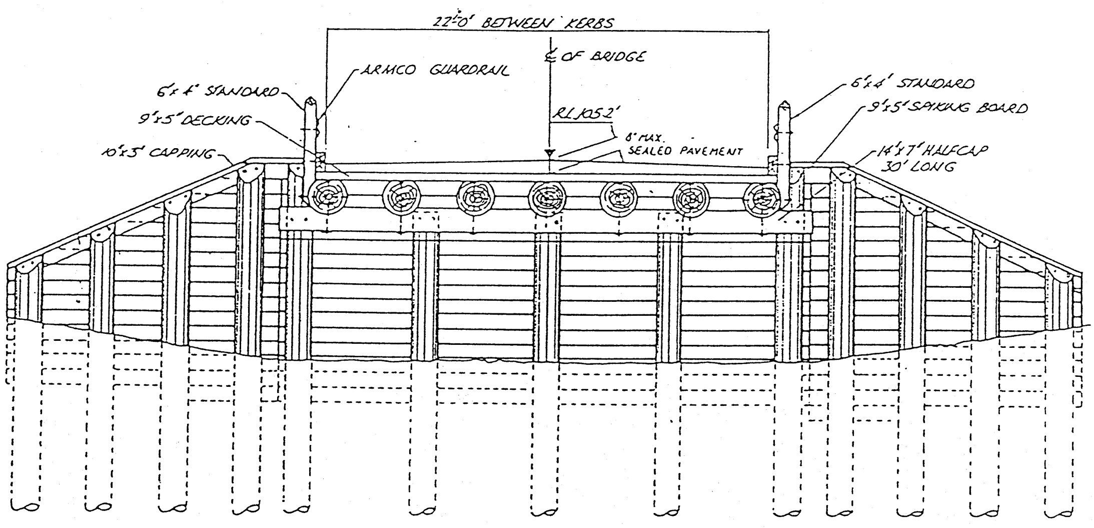
APPENDIX A

TYPICAL TIMBER BRIDGE DETAILS

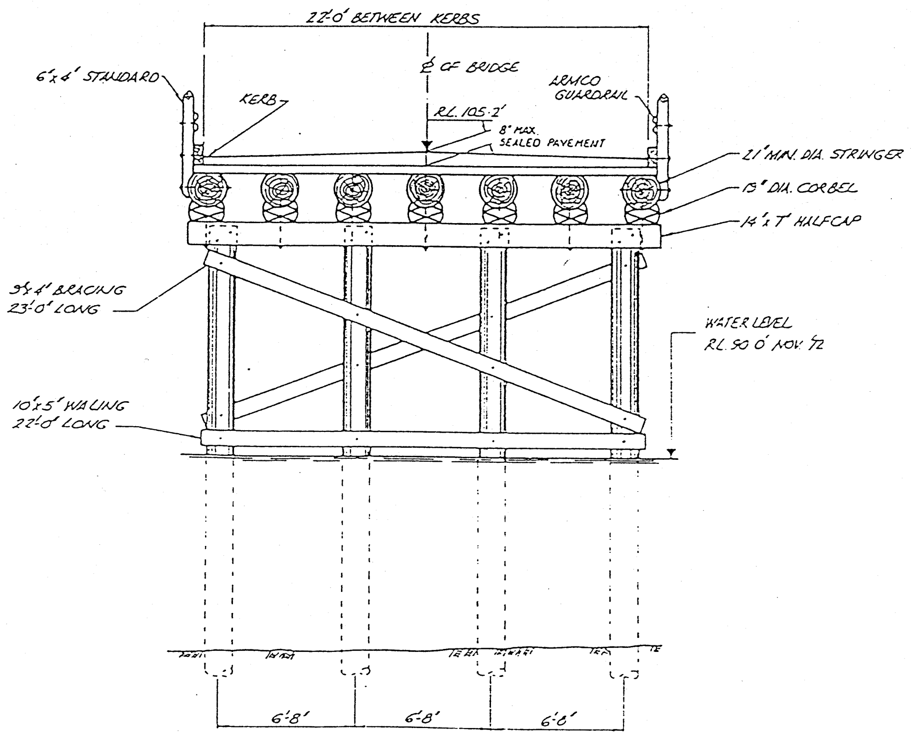


TYPICAL BRIDGE ARRANGEMENT

FIGURE A



SECTIONAL ELEVATION - ABUTMENT



— SECTIONAL ELEVATION - PIER —
TYPICAL

TYPICAL ABUTMENT & PIER DETAILS

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APPENDIX B

WORKING STRESS DESIGN GUIDELINES

USER NOTE:

These guidelines have been based primarily on the superseded working stress Timber Structures Code AS 1720.1-1988. This guideline does not replace the superseded code as there are a number of omissions and modifications such that the guidelines are specific to MRWA standard timber bridges. The replacement ultimate limit state Timber Structures Code AS 1720.1-1997 is considered not entirely suitable by Structures Engineering for the use on timber bridge structures as it would appear to give overly conservative load limitations compared to standards that have been used successfully by MRWA over many years.

Consultation with experts in this field, including a member of the committee for the new code, confirmed that the new code ultimate limit state code was intended to provide outcomes that were equivalent to the previous working stress code. However a comprehensive comparison of the two codes found this not to be the case for timber bridges. Some of the main factors causing the inconsistency between the working stress code and the ultimate limit state code are as follow:

- Ultimate load factors used for Dead Load, Superimposed Dead Load and Live Load.
- Conservative capacity reduction factors adopted in the Ultimate Limit State (ULS) code when considered in conjunction with the Ultimate Load factors adopted.

Adoption of the new code would have resulted in timber bridges being rated with reduced capacities and would have also resulted in a lower maintenance intervention level. This was considered incorrect and as a consequence, MRWA produced this Load Rating and Refurbishment Design Manual.

Engineering judgment should be applied when using these guidelines.

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B1 SCOPE AND GENERAL

B1.1 Scope

This manual sets out the design methods for the structural use of timber, which are based on the principles of structural mechanics and on data established by research. It is intended for use in the design or appraisal of structural elements comprised of timber. To this end, it provides design data for sawn timber, timber in pole form and various types of fastenings.

For ease of use the simpler design situations are covered in the main body of the text.

B1.2 New Materials and Methods

This manual shall not be interpreted to prevent the use of material or of methods of design or construction not specifically referred to herein. Nor is the classification of timbers into strength groups (Clause B1.3) or their grouping for joint design (Clause B4) to be interpreted as precluding the use of design stresses or other design data derived for a particular timber or grade of timber on the basis of authoritative research information.

B1.3 Timber Quality

All timber used shall comply with the requirements of AS 2082-2000. The following points shall be noted:

a) General
Tables B2.1 and B2.2 herein list common species used for structural purposes together with their strength classifications and design density.

b) Timber classification
Timber species are classified into seven strength groups S1 to S7 in the unseasoned condition and eight strength groups SD1 to SD8 in the seasoned condition. The timber species are also classified into six joint groups J1 to J6 if used unseasoned, and JD1 to JD6 if used seasoned. Sawn structural timber, pole timbers and plywood are classified into 12 stress grades F2 to F34 when these have been graded according to the appropriate grading Standard or other approved specification.

c) Stress grade and species identification
Structural timber used shall have its stress grade identified. For many purposes it may also be necessary to specify a particular species. When a particular species is specified the specification shall require that all pieces of timber be suitably identified as to the species.

Notes:

1. The design properties recommended have been chosen on the assumption that structures of unseasoned timber that are allowed to dry will not receive their full design load until a period of air drying for at least two weeks has taken place. Freshly sawn timber that is unseasoned, or has recently been treated with waterborne chemicals, tends to have a reduced resistance and stiffness to sustained loads during the initial drying period.
2. Usually, only a limited number of the timber species and stress grades listed will be readily available at any particular place and time.

d) Change of grade or durability

Care shall be taken to account for any change in original grading or preservative treatment as a result of sawing or dressing. Regrading will be necessary if members are longitudinally re-sawn. Machining may remove preservative envelopes rendering the treatment ineffective.

e) Treated timber

Timber, treated by impregnation with waterborne chemicals such as preservatives, is classified as unseasoned timber unless seasoning is specified.

B1.4 General Design Considerations

B1.4.1 Loads

B1.4.1.1 General

A structure, and any part of a structure, shall be designed for the loads specified as is appropriate to the end-use of a specific structure or part of a structure.

B1.4.1.2 Load Duration

The significance of duration of loading in the design of timber structures shall be noted and particular attention paid to the term 'duration of loading'. For the definition of this term see Clause B1.6.1 and for further information Clause B2.5.1.

B1.4.2 Design Methods

B1.4.2.1 General

A structure, or part of a structure, or an individual structural element shall be capable of sustaining the most adverse likely combination of loads. Every part of the structure shall be proportioned so that the permissible stresses determined are not exceeded.

B1.4.2.2 Stress Analysis

All stresses shall be calculated on the basis of elastic theory in order that the stresses may be satisfied with regard to the load effects at any particular location. For example the applied bending movement M and force V on a beam of rectangular cross-section shall be checked by-

$$M \leq (bd^2/6)F_b \qquad \text{(B1.1)}$$

$$V \leq (2bd/3)F_s \qquad \text{(B1.2)}$$

where

b and d = the breadth and depth of the member

F_b and F_s = the permissible design stresses in bending and shear

B1.4.3 Other Design Considerations

B1.4.3.1 Stability

The stability of the structure as a whole shall be investigated, and mass and anchorage shall be provided so that the structure is in overall equilibrium.

B1.4.3.2 Buckling Restraints

Where there may be some doubt as to the effectiveness of buckling restraints, appropriate computations shall be made to check the stiffness and strength of the restraints.

B1.4.3.3 Erection and Other Extraneous Forces

Adequate provision shall be made to resist the lateral and other forces that can occur during the transport of structural elements, and during and after the erection of a structure.

B1.4.3.4 Secondary Stresses

Careful consideration shall be given to possible secondary stresses. Where these cannot be reduced to negligible proportions, suitable provisions in the design or some reduction in permissible primary stresses shall be made.

B1.4.3.5 Shrinkage

When using unseasoned timber, consideration shall be given to the effects of shrinkage. Detailing of the joints shall not restrain shrinkage where splitting could render the joint ineffective. Consideration shall also be given to architectural detailing to avoid damage or unsightly appearances resulting from differential movement on structural members caused by timber shrinkage. These comments also apply to timber, which has been impregnated with waterborne chemicals, which has not been re-seasoned after treatment.

Note:

> For most timbers the magnitude of shrinkage is in the range of 0.1% to 0.3% in the direction of the wood grain and 2% to 10% transverse to this direction.

B1.4.3.6 Deformations

Timber structures shall be designed so that deformations occuring in-service do not impair the strength and serviceability of the structures or any part thereof, nor cause damage to other structural components. Timber members shall have sufficient stiffness so that undesirable deflections and vibrations are avoided.

Notes:

1. The responsibility for deflection and stiffness limits should rest with the design engineer.
2. In computing design deflections, it should be appreciated that timber is variable with respect to its structural properties. It should also be noted that the moduli given in Table B2.3 refer to design values for groups of timber. If for some reason (e.g. to specify camber) accurate predictions of deflection are required, detailed information relevant to specific species of timber under consideration should be used.

B1.4.3.7 Timber Dimensions for Engineering Calculations

All engineering calculations shall be based on the minimum net cross-section. Such calculations shall not be based on the nominal cross-section.

B1.4.3.8 Timber in Natural Pole Form

For logs or poles complying with the quality requirements of AS 2209, the correspondence between strength groups and stress grades is as shown in Table B5.1.

B1.4.3.9 Biological Deterioration

Generally, timber under cover and in well-ventilated conditions and not in contact with the ground or free water, is not subject to fungal attack. However, such timber may be subject to termite attack and to attack by other insects in parts of Australia. If conditions favourable for biological attack exist, then steps shall be taken to eliminate the hazards. This is particularly important in structures where there is no load sharing capacity.

B1.5 Workmanship and Maintenance

B1.5.1 General

The following requirements are intended to help insure that a structure or element when fabricated performs, and will continue to perform, structurally in the manner intended by the designer of the structure.

B1.5.2 Moisture Content

When structures or elements are to be fabricated with seasoned timber in situations where dimensional stability is critical, the designer of the structure shall ascertain the average equilibrium moisture content for the environment in which the structures or elements are to be erected, and shall specify that each piece of timber used shall have an average moisture content at the time of fabrication that is within 3 percent of the equilibrium value.

B1.5.3 Corrosion

The designer of the structure shall take due account of any possible corrosive effects on metal connectors.

B1.5.4 Maintenance

Where in the opinion of the designer of a structure special maintenance is required for a structure to fulfil its intended function, then such maintenance shall be specified in relevant documents.

B1.6 Definitions

B1.6.1 Technical Definitions

Basic working stress – stress appropriate to an arbitrarily chosen, but constant, basic reference set of conditions. It is derived from the known strength properties of timber, due allowance having been made for such factors as material variability, long-duration loading, grade of timber and a safety factor.

Basic working load for connectors – load appropriate to an arbitrarily chosen, but constant, basic reference set of conditions. It is derived from the known strength properties of the timber-connector system, due allowance having been made for such factors as material variability, long-duration loading, grade of timber, and a safety factor.

Collapse-susceptible timber – timber for which the shrinkage values before and after conditioning differ by more than 2 percent.

Corewood – timber adjacent to or including pith, that is of density less than 80 percent of the density of mature trees.

Duration of loading – period during which a member, a structural element or a complete structure is stressed as a consequence of the loads applied.

In-grade testing – procedure for testing and evaluating stress graded, structural size timber.

Partially seasoned timber – wood which has a average moisture content of greater than 15 percent but does exceed 25 percent.

Permissible stress – maximum stress to be used in the design of an element of a structure. It is obtained from the basic working stress appropriately modified for the type of structure and service conditions.

Seasoned timber – wood in which the maximum moisture content anywhere within a piece does not exceed 15 percent.
Note:

Seasoned timber is sometimes referred to as ‘dry’ or ‘air dried’ timber. It includes kiln-dried timber.

Stress Grade – classification of timber for structural purposes by means of either visual or machine grading to indicate the basic working stresses and stiffnesses to be used for structural design purposes.
Note:

The stress grade is designated in a form such as ‘F7’ which indicates that, for such a grade of material, the basic working stress in bending is approximately 7 MPa.

Unseasoned Timber – wood in which the average moisture content of each piece exceeds 25 percent.
Note:

Unseasoned timber is sometimes referred to as ‘green’ timber.

B1.7 Units

Unless otherwise stated, the units of measurement used in this Standard are in accordance with the International System of Units (SI).
Note:

In general N (newton), mm (millimetre) and MPa (megapascal) are appropriate units to be used.

B2 BASIC PROPERTIES OF STRUCTURAL TIMBER

B2.1 General

Permissible stresses for structural timber shall be obtained though modifying basic working stress values by factors appropriate to the service conditions. This general procedure applies to all types of structural timber, including sawn timber, laminated timber, natural round timber and plywood.

B2.2 Structural Classifications

Tables B2.1 and B2.2 list the structural classifications and design densities (for computing dead loads) of timber species and species groups that are commonly used in Australia.

The data given in Tables B2.1 and B2.2 are taken from various standards. In particular AS 2082, AS 2209, AS/NZS 2269, AS 2858 and AS/NZS 2878; any changes to these Standards shall be taken to supersede the data cited. In addition, any stress grades evaluated through in-grade testing of full-size structural material shall be taken to supersede all the above information, providing that at least 5% of all elements being evaluated are in-grade tested.

Notes:
1. The density of unseasoned timber depends on its moisture content that reduces as the timber dries. The values given in Tables B2.1 and B2.2 have been computed on the basis that the percentage saturation of the timber is 45 and 80 percent for softwoods and hardwoods respectively.
2. The values of density given in Tables B2.1 and B2.2 do not represent average values for the species indicated; they are intended for use in computing the dead load imposed by the timber.
3. The moduli of elasticity given in Table B2.3 are intended to represent average values except where species mixtures or species with high variability are concerned; in the latter case, the cited moduli of elasticity are less than the average values.

B2.3 Basic Working Stresses and Modulus of Elasticity

B2.3.1 Basic Working Stresses Parallel to Grain and Shear Stresses in Beams

These basic working stresses are given in Table B2.3 for the various stress grades.

B2.3.2 Basic Working Stress in Compression Perpendicular to the Grain and Shear Stress at Joint Details

These basic working stresses are given in Table B2.4 for each strength group and are applicable to all stress grades within the strength group.

B2.3.3 Basic Working Stress in Compression at an Angle to the Grain

The basic working stresses in compression at angles to the grain other than 0° and 90° shall be denoted by $F'_c\theta$ and shall be calculated from the Hankinson formula:

$$F'_c\theta = (F'_c \times F'_p) / (F'c \sin^2 \theta + F'_p \cos^2 \theta) \qquad \text{(B2.1)}$$

where θ is the angle between the direction of the load and the direction of the grain.

B2.3.4 Modulus of Elasticity and Rigidity

Design values of the modulus of elasticity and rigidity are given in Table B2.3.

B2.4 Design

B2.4.1 Permissible Stresses

Permissible stresses for structural timber, whether sawn or laminated construction, or in pole form, shall be obtained by multiplying the basic working stresses given in Clause B2.3 by modification factors such as those given in Clause B2.5 as are appropriate to the service conditions. For example, F_b the permissible stress in bending is given by –

$$F_b = k\,F'_b \qquad \text{(B2.2)}$$

where

k = the product of the relevant modification factors, such as those in Clause B2.5, as are appropriate to the particular service conditions for which the structural member is being designed.

Note: As an example, the factor k for the design bending stress of a solid timber beam is typically given by $k = k_1\,k_2\,k_8\,k_{11}\,k_{12}$.

B2.4.2 Deflections

Deflection calculations shall take into account the modification factors in Clause B2.5.1.2.

Table B2.1 - STRENGTH CLASSIFICATIONS AND DESIGN DENSITIES FOR SOME COMMON GROUPS OF TIMBER

Species Group	Moisture Condition	Strength Group	Joint Group	Stress Grade							Design Density kg/m^3
				Structural Timber							
				Structural No. 1	Structural No. 2	Structural No. 3	Structural No. 4	Structural No. 5	Structural Plywood	Pole Timber	
Mixed Australian hardwoods (excluding rainforest species) from SA and southern NSW	Unseasoned	S4	J3	F14	F11	F8	F7	--	--	F11	1050
	Seasoned	SD4	JD3	F22	F17	F14	F11	--	F17	--	650
Ash-type Eucalypts from NSW Highlands, Victoria and Tasmania	Unseasoned	S4	J3	F14	F11	F8	F7	--	--	F17	1050
	Seasoned	SD4	JD3	F22	F17	F14	F11	--	F17	--	650
Non ash-type Eucalypts from Queensland and NSW	Unseasoned	S3	J2	F17	F14	F11	F8	--	--	F22	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	--	750
Rainforest species	Unseasoned	S7	J4	F7	F5	F4	--	--	--	F8	800
	Seasoned	SD7	JD4	F11	F8	F7	F5	--	F8	--	500
Mixed pinus species (Australian Grown)	Unseasoned	--	--	--	--	--	--	--	--	--	850
	Seasoned	SD7	JD4	F11	F8	F7	F5	--	F8	--	550
Mixed softwood species (excluding pinus species)	Unseasoned	--	--	--	--	- -	--	--	--	--	850
	Seasoned	SD8	JD4	F8	F7	F5	--	--	F7	--	500
Imported softwoods (unidentified)	Unseasoned	S7	J6	F7	F5	F4	--	--	--	--	850
	Seasoned	SD8	JD6	F8	F7	F5	F4	--	F7	--	400

Table B2.2 - STRENGTH CLASSIFICATIONS AND DESIGN DENSITIES FOR SOME COMMON SPECIES OF TIMBER

Species Group	Moisture Condition	Strength Group	Joint Group	Stress Grade							Design Density kg/m^3
				Structural Timber					Structural Plywood	Pole Timber	
				Structural No. 1	Structural No. 2	Structural No. 3	Structural No. 4	Structural No. 5			
Ash alpine	Unseasoned	S4	J3	F14	F11	F8	F7	--	--	F17	1050
	Seasoned	SD4	JD3	F22	F17	F14	F11	--	F17	--	650
Ash mountain	Unseasoned	S4	J3	F14	F11	F8	F7	--	--	F17	1050
	Seasoned	SD3	JD3	F27	F22	F17	F14	--	F22	--	650
Ash silvertop	Unseasoned	S3	J2	F17	F14	F11	F8	--	--	F22	1100
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	--	850
Balau	Unseasoned	S2	J2	F22	F17	F14	F11	--	--	F27	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	--	900
Blackbutt	Unseasoned	S2	J2	F22	F17	F14	F11	--	--	F27	1150
	Seasoned	SD2	JD2	F34	F27	F22	F17	--	F27	--	900
Box brush	Unseasoned	S3	J2	F17	F14	F11	F8	--	--	F22	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	--	900
Box grey coast	Unseasoned	S1	J1	F27	F22	F17	F14	--	--	F34	1200
	Seasoned	SD1	JD1	F34	F34	F27	F22	--	F34	--	1100
Brown barrel	Unseasoned	S4	J3	F14	F11	F8	F7	--	--	F17	1100
	Seasoned	SD4	JD3	F22	F17	F14	F11	--	F17	--	750
Chengal	Unseasoned	S1	J2	F27	F22	F17	F14	--	--	F34	1150
	Seasoned	SD2	JD2	F34	F27	F22	F17	--	F27	--	950
Fir Douglas North America	Unseasoned	S5	J4	F11	F8	F7	F5	F4	--	F14	710
	Seasoned	SD5	JD4	F14	F11	F8	F7	F5	F14	--	550
Fir Douglas Elsewhere	Unseasoned	S6	J5	F8	F7	F5	F4	--	--	F11	710
	Seasoned	SD6	JD5	F14	F11	F8	F7	F5	F11	--	550
Gum blue southern	Unseasoned	S3	J2	F17	F14	F11	F8	--	--	F22	1150
	Seasoned	SD2	JD2	F34	F27	F22	F17	--	F27	--	1000
Gum blue Sydney	Unseasoned	S3	J2	F22	F17	F14	F11	--	--	F27	1100
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	--	850
Gum Red River	Unseasoned	S5	J2	F11	F8	F7	F5	--	--	F14	1150
	Seasoned	SD5	JD2	F17	F14	F11	F8	--	F14	--	900
Gum rose	Unseasoned	S3	J2	F17	F14	F11	F8	--	--	F22	1100
	Seasoned	SD4	JD2	F22	F17	F14	F11	--	F17	--	750

Table B2.2 - CONTINUED

Species Group	Moisture Condition	Strength Group	Joint Group	Stress Grade							Design Density kg/m³
				Structural Timber					Structural Plywood	Pole Timber	
				Structural No. 1	Structural No. 2	Structural No. 3	Structural No. 4	Structural No. 5			
Gum Spotted	Unseasoned	S2	J1	F22	F17	F14	F11	--	---	F27	1200
	Seasoned	SD2	JD1	F34	F27	F22	F17	--	F27	---	1100
Hardwood, Johnstone River	Unseasoned	S2	J1	F22	F17	F14	F11	--	---	F27	1150
	Seasoned	SD3	JD1	F27	F22	F17	F14	--	F22	---	950
Hemlock Western	Unseasoned	S6	J4	F8	F7	F5	F4	--	---	F11	750
	Seasoned	SD6	JD4	F14	F11	F8	F7	F5	F11	---	500
Hem-fir	Unseasoned	S7	J5	F7	F5	F4	--	--	--	F8	750
	Seasoned	SD7	JD5	F11	F8	F7	F5	F4	F8	---	550
Ironbark, Grey	Unseasoned	S1	J1	F27	F22	F17	F14	--	---	F34	1250
	Seasoned	SD1	JD1	---	F34	F27	F22	--	F34	---	1100
Ironbark, Red Narrow Leaved	Unseasoned	S2	J1	F22	F17	F14	F11	--	---	F27	1250
	Seasoned	SD3	JD1	F27	F22	F17	F14	--	F22	---	1050
Jarrah	Unseasoned	S4	J2	F14	F11	F8	F7	--	---	F17	1100
	Seasoned	SD4	JD2	F22	F17	F14	F11	--	F17	---	800
Kapur	Unseasoned	S3	J2	F17	F14	F11	F8	--	---	F22	1100
	Seasoned	SD4	JD2	F22	F17	F14	F11	--	F22	---	750
Karri	Unseasoned	S3	J2	F17	F14	F11	F8	--	---	F22	1150
	Seasoned	SD2	JD2	F34	F27	F22	F17	--	F27	---	900
Kempas	Unseasoned	S2	J1	F22	F17	F14	F11	--	---	F27	1100
	Seasoned	SD2	JD2	F34	F27	F22	F17	--	F27	---	900
Kwila (Merbau)	Unseasoned	S2	J2	F22	F17	F14	F11	--	---	F27	1150
	Seasoned	SD3	JD2	F34	F27	F22	F17	--	F27	---	850
Lumbaya, Chengkulang	Unseasoned	S5	J3	F11	F8	F7	F5	--	---	F14	1100
	Seasoned	SD5	JD3	F17	F14	F11	F8	--	F14	---	750
Mahogany, red	Unseasoned	S2	J1	F22	F17	F14	F11	--	---	F27	1200
	Seasoned	SD3	JD1	F27	F22	F17	F14	--	F22	---	950
Marri	Unseasoned	S3	J2	F17	F14	F11	F8	--	---	F22	1100
	Seasoned	SD3	JD2	F27	F22	F17	F14	--	F22	---	850
Meranti dark red	Unseasoned	S5	J4	F11	F8	F7	F5	--	---	F14	1100
	Seasoned	SD6	JD4	F14	F11	F8	F7	--	F11	---	600-750

Table B2.2 - CONTINUED

Species Group	Moisture Condition	Strength Group	Joint Group	Stress Grade							Design Density kg/m³
				Structural Timber					Structural Plywood	Pole Timber	
				Structural No. 1	Structural No. 2	Structural No. 3	Structural No. 4	Structural No. 5			
Mersawa	Unseasoned	S6	J3	F8	F7	F5	F4	--	---	F11	1050
	Seasoned	SD6	JD3	F14	F11	F8	F7	---	F11	---	700
Messmate	Unseasoned	S3	J3	F17	F14	F11	F8	---	---	F22	1100
	Seasoned	SD3	JD3	F27	F22	F17	F14	---	F22	---	750
Oak,.tulip brown	Unseasoned	S2	J2	F22	F17	F14	F11	---	---	F27	1150
	Seasoned	SD2	JD2	F34	F27	F22	F17	---	F27	--	900
Pine, Cypress, white	Unseasoned	S5	J3	---	---	F7	F5	F4	---	F14	850
	Seasoned	SD6	JD3	---	---	---	---	---	F11	---	700
Pine,.hoop	Unseasoned	S6	J4	F8	F7	F5	F4	---	---	F11	800
	Seasoned	SD3	JD4	F17	F14	F11	F8	F7	F14	---	550
Pine, radiata (Australia and New Zealand)	Unseasoned	S6	J4	---	---	---	---	---	---	F11	800
	Seasoned	SD6	JD4	F14	F11	F8	F7	F5	F11	---	550
Pine, slash	Unseasoned	S5	J3	F11	F8	F7	F5	F4	---	F14	850
	Seasoned	SD5	JD3	F17	F14	F11	F8	F7	F14	---	650
Spruce-pine-fir	Unseasoned	---	---	---	---	---	---	---	--	---	700
	Seasoned	SD7	JD5	F8	F8	F7	F5	F4	F8	---	500
Stringybark brown	Unseasoned	S3	J2	F17	F14	F11	F8	--	---	F22	1100
	Seasoned	SD3	JD2	F27	F22	F17	F14	---	F22	---	850
Stringybark yellow	Unseasoned	S3	J2	F17	F14	F11	F8	---	---	F22	1150
	Seasoned	SD3	JD2	F27	F22	F17	F14	---	F22	---	900
Tallowwood	Unseasoned	S2	J1	F22	F17	F14	F11	---	---	F27	1200
	Seasoned	SD2	JD2	F34	F27	F22	F17	---	F27	---	1000
Turpentine	Unseasoned	S3	J2	F17	F14	F11	F8	---	---	F22	1050
	Seasoned	SD3	JD2	F27	F22	F17	F14	---	F22	---	950
Wandoo	Unseasoned	S2	J1	F22	F17	F14	F11	---	---	F27	1250
	Seasoned	SD3	JD1	F27	F22	F17	F14	---	F22	---	1100

Table B2.3 - BASIC WORKING STRESSES AND STIFFNESS FOR STRUCTURAL TIMBER

Stress Grade	Basic working stress, MPa				Short duration modulus of elasticity* (E)	Short duration modulus of rigidity (G)
	Bending (F'_b)	Tension parallel to grain (F'_t)	Shear in beams (F'_s)	Compression parallel to grain (F'_c)		
F34	34.5	20.7	2.45	26.0	21500	1430
F27	27.5	16.5	2.05	20.5	18500	1230
F22	22.0	13.2	1.70	16.5	16000	1070
F17	17.0	10.2	1.45	13.0	14000	930
F14	14.0	8.4	1.25	10.2	12000	800
F11	11.0	6.6	1.05	8.4	10500	700
F8	8.6	5.2	0.85	6.6	9100	610
F7	6.9	4.1	0.70	5.2	7900	530
F5	5.5	3.3	0.60	4.1	6900	460
F4	4.3	2.6	0.50	3.3	6100	410
F3	3.4	2.0	0.45	2.6	5200	350
F2	2.7	1.6	0.35	2.1	4500	300

* The modulus of elasticity includes an allowance of about 5 percent for shear deformation.

Table B2.4 - BASIC WORKING STRESSES FOR COMPRESSION PERPENDICULAR TO GRAIN AND SHEAR AT JOINTS

Strength		Basic working stress, MPa	
Unseasoned	Seasoned	Compression perpendicular to grain (F'_p)	Shear at joints details (F'_{sj})
-	SD1	10.4	4.15
-	SD2	9.0	3.45
-	SD3	7.8	2.95
S1	SD4	6.6	2.45
S2	SD5	5.2	2.05
S3	SD6	4.1	1.70
S4	SD7	3.3	1.45
S5	SD8	2.6	1.25
S6	-	2.1	1.05
S7	-	1.7	0.85

B2.5 Modification Factors

B2.5.1 Duration of Load

B2.5.1.1 Effect on Strength

In order to derive permissible design stresses, the basic working stress shall be multiplied by the appropriate duration of load factor k_1 from Table B2.5.

In checking the strength of a structural element, all load combinations must be considered. For any given combination of loads of differing duration, the factor k_1 to be used is that appropriate to the load that is of the shortest duration. In Table B2.5, the effective duration of a peak load refers to the cumulative duration for which the peak load occurs.

For the purposes of interpretation in the selection of load-duration factors in this Standard, the following shall apply:

(a) Dead loads, and live loads which are removed or replaced at regular intervals such that the structure remains fully loaded for a substantial portion of its life, are to be considered 'permanent loads'.

(b) Live Loads (such as those due to vehicles or people) that act on floors or decks and are applied at frequent but irregular intervals such that the structure is unloaded, or loaded well below the allowable maximum, for most of each day, are to be considered 'loads of five months duration'.

(c) Live Loads, such as those arising during erection and maintenance, and at infrequent crowd loading, applied for periods of a few days and at infrequent intervals, are to be considered 'loads of five second duration'.

Table B2.5 - DURATION OF LOAD FACTOR FOR STRENGTH (k_1)

Type of load	Effective duration of peak load	Multiplying factor (k_1)	
		Basic stresses for solid timber	Basic working loads for laterally loaded connectors*
Instantaneous	5 seconds	1.75	2.00
Standard test	5 minutes	1.75	1.75
Short term	5 hours	1.70	1.50
Medium term	5 days	1.65	1.35
Long term	5 months	1.40	1.20
Permanent	50 years	1.00	1.00

* For connectors loaded in withdrawal (pull out) and for the strength of steel in connectors, k_1 = 1.00.

B2.5.1.2 Effect on Stiffness

For members in bending and compression or for members in tension, the calculated short-term deformation shall be multiplied by the appropriate creep factor j_2 or j_3, as given in Table B2.6 and illustrated graphically in Figures B2.1 and B2.2.

Values intermediate between those given in Table B2.6 may be obtained through an interpolation involving the logarithm of time, and linear function of initial moisture content as shown in Figures B2.1 and B2.2.

When several types of load act on a timber member, the maximum deformation shall be taken to be equal to the sum of the deformations computed for each type of load acting alone.

The modification factors j_2 and j_3 given in Table B2.6 are not applicable to collapse susceptible hardwoods (see Clause B1.6.1) when their initial moisture content is above 25%. For these timbers the creep factors may be considerably greater than the values shown.

Notes:

1. The loads to be considered in computing deflections are not only the peak loads used for strength checks, but also all loads that act during the life of the structure. In general, peak values of live load are not of a permanent nature; accordingly if the designer wishes to compute the long term deformations of a structure he/she must first estimate the portion of the load that is permanently or semi-permanently applied, and then use an appropriate creep factor.
2. Where there is a recovery period of more than ten times that of the applied load, the creep component of deformation may be assumed to be totally recovered.

Table B2.6 - DURATION OF LOAD FACTOR FOR DEFLECTION

Initial moisture content* (%)	For bending, compression and shear member (j_2)		For tension members (j_3)	
	Load duration ≤ 1 day	Load duration ≥ 1 year	Load duration ≤ 1 day	Load duration ≥ 1 year
≤ 15	1	2	1	1
≥ 25	1	3	1	1.5

*Moisture content at the time of load application.

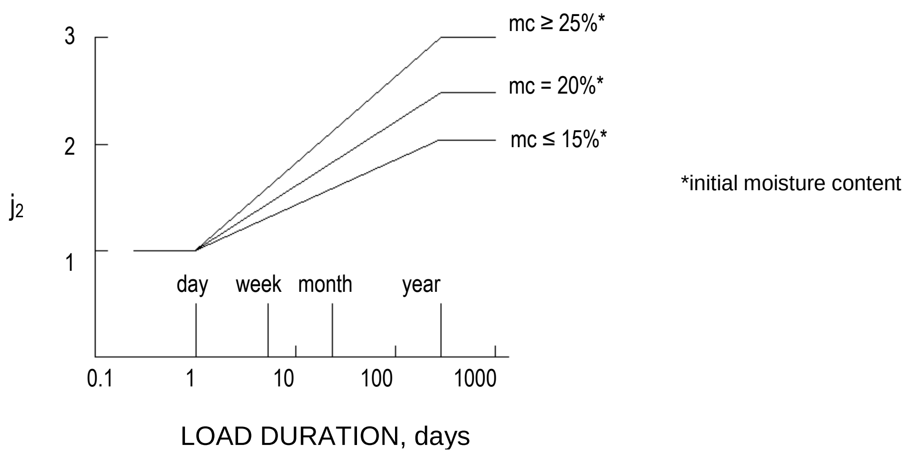


Figure B2-1 - DURATION FACTOR j_2 FOR BENDING AND COMPRESSION DEFORMATIONS

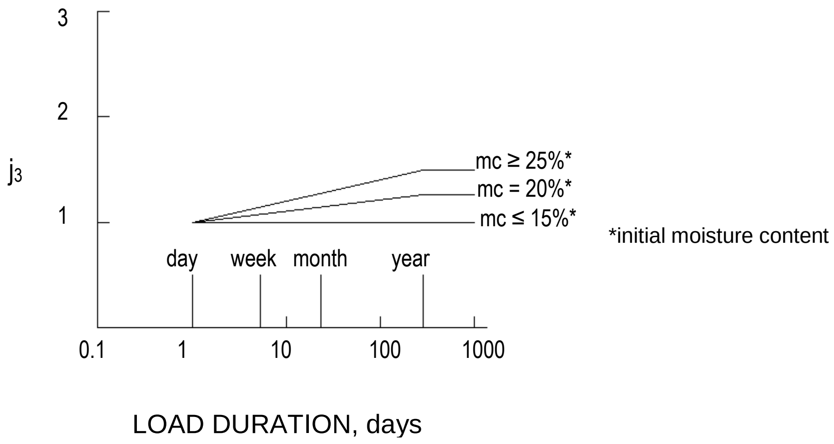


Figure B2-2 - DURATION FACTOR j_3 FOR TENSION DEFORMATIONS

B2.5.2 Moisture Content

Depending on the initial moisture content of the timber and the moisture content at time of loading and throughout its life, the basic working stresses shall be modified as follows:

(a) Unseasoned timber: Where unseasoned timber is used, the basic working stresses shall be those in Tables B2.3 and B2.4 appropriate to the stress grade and strength group of the unseasoned timber as indicated in Tables B2.1 and B2.2.

(b) Unseasoned timber partly dry before use: Where unseasoned timber is used under normal conditions of temperature and humidity and will not be subject to its full design load until it has partly seasoned, i.e. to below 25% moisture content, the basic working stresses for unseasoned timber may be increased by multiplying by the factor k_4 given in Table B2.7.

Table B2.7 - PARTIAL SEASONING FACTOR

Least dimension of member	38 mm or less	50 mm	75 mm	100 mm or more
Value of k_4	1.15	1.10	1.05	1.00

(c) Seasoned Timber

(i) Where seasoned timber is used, the basic working stresses shall be those in Tables B2.3 and B2.4 appropriate to the stress grade and strength group of the timber in the seasoned condition as indicated in Tables B2.1 and B2.2.

(ii) Where seasoned timber is subjected to conditions in which its average moisture content for a 12-month period is expected to exceed 15%, the basic working stresses shall be decreased by multiplying by the factor k_5 determined as the greater of:

$$k_5 = 1 - \left[\frac{EMC - 15}{10}\right]\left[1 - \frac{F'(unseasoned)}{F'(seasoned)}\right] \qquad \text{(B2.3(a))}$$

and

$$k_5 = \left[\frac{F'(unseasoned)}{F'(seasoned)}\right] \qquad \text{(B2.3(b))}$$

where

EMC	The highest value of the annual average moisture content (percent) that the timber will attain in service.
F' (seasoned)	The basic working stress for the seasoned material.
F' (unseasoned)	The basic working stress for material of the same grade in the unseasoned condition.

B2.5.3 Temperature

For covered timber structures under ambient conditions, no modification to the basic working stresses need be made for the effect of temperature except that where seasoned timber is used in structures erected in the coastal regions of Queensland north of latitude 25°S and all other regions of Australia north of latitude 16°S, the basic working stresses shall be multiplied by a factor k_6 of 0.9.

B2.5.4 Length and Position of Bearing

For rectangular bearing areas for bearings of length less than 150 mm and with the bearing surface 75 mm or more from the end of a piece of timber, the basic working stress in bearing perpendicular to the grain given in Table B2.4 may be multiplied by the appropriate factor k_7 in Table B2.8, the length of bearing being measured parallel to the grain of the loaded member.

For circular bearing areas the effective bearing length shall be taken as being equal to the diameter of the bearing area.

Table B2.8 - LENGTH OF BEARING FACTOR

Length of bearing of member (mm)	12	25	50	75	125	150 or more
Value of k_7	1.85	1.60	1.30	1.15	1.05	1.00

B2.5.5 Load Sharing

B2.5.5.1 General

When a structural system consists of parallel acting elements that interact to assist each other then the basic working stresses may be increased by the appropriate load sharing factor.

B2.5.5.2 Parallel Structural Systems

For structural systems comprised of two or more elements effectively connected so that all of the elements are constrained to the same deformation, the load sharing factor k_8 may be obtained from Table B2.9, and applied to the basic working stresses for bending and compression. If the effective number of elements is not an exact integer, then a suitable value of k_8 may be derived by linear interpolation.

Table B2.9 - PARALLEL SUPPORT FACTOR

Effective number of elements carrying common load (n_{eff})	**Factor k_8**
1	1.00
2	1.14
3	1.20
4	1.24
5	1.26
6	1.28
7	1.30
8	1.31
9	1.32
10 or more	1.33

B2.5.5.3 Grid Systems

Where constructions are such that three or more members act together to support an overlying set of members usually laid at right angles to the supporting members, a load-sharing factor k_9 may be applied to the basic working stress for bending, to the supporting members. This factor is given by the equation:

$$k_9 = k_0 + (k_8 - k_0)\,[1.0 - 2\,(s / L)\,] \qquad \text{(B2.4)}$$

but not less than k_0, where

- s = the centre-to-centre spacing of the supporting members
- L = span of the supporting member
- k_8 = the load sharing factor for parallel structural systems (see Clause B2.5.5.2)
- k_0 = 1.0 for solid timber

The load sharing factor k_9, is illustrated graphically in Figure B2.3.

Note: In addition to load sharing characteristics, grid systems also provide a method for laterally distributing concentrated loads.

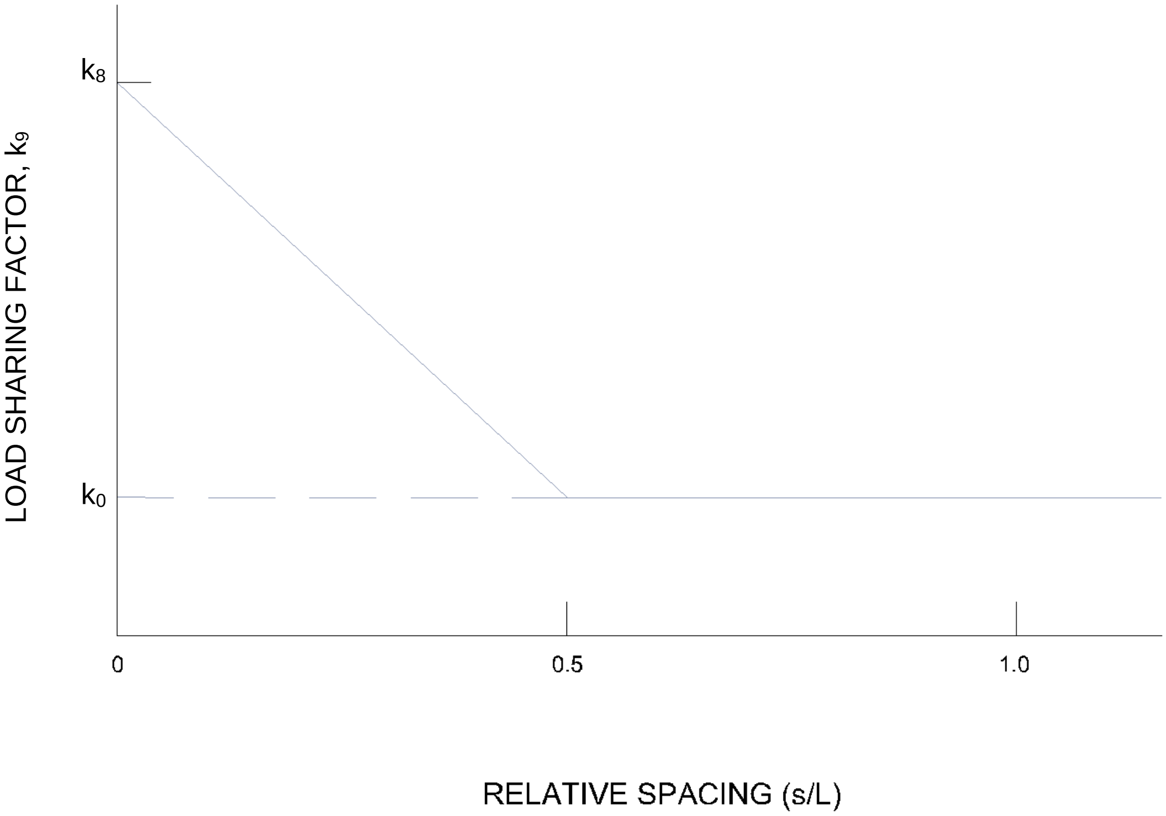


Figure B2-3 - LOAD SHARING FACTOR k_9 FOR GRID SYSTEMS

B2.5.6 Size Factor for Flexural and Tension Members

The basic working stress in bending and tension shall be multiplied by the size factor k_{11} as given in Table B2.10. Linear interpolation may be used for intermediate sizes.

For beams of depth d greater than 1500 mm, the value of k_{11} shall be taken to be given by:

$$k_{11} = (300 / d)^{0.167} \qquad \text{(B2.5)}$$

Table B2.10 - SIZE FACTORS FOR BEAMS AND TENSION MEMBERS

Maximum depth of beam or twice width of tension member (mm)	300	375	500	625	750	1 000	1 250	1 500
Value of k_{11}	1.00	0.96	0.92	0.89	0.86	0.82	0.79	0.77

B2.5.7 Stability Factor

In the design of slender structural members, a factor k_{12} is used to allow for the effects of slenderness on strength. It is defined by:

$$F = k_{12}F_0 \qquad \text{(B2.6)}$$

where

F = the nominal design stress

F_0 = the value of F if the structural member were completely stable

The factor k_{12} depends on both material factors and the slenderness coefficient S. These factors and the slenderness coefficient are defined for each type of slender structural member in the section of this Guideline appropriate to that element.

B2.5.8 Material and Application Factor

For all timber members, the basic working stresses given in Tables B2.3 and B2.4 shall be multiplied by the material and application factor k_2 shown in Table B2.11.

Table B2.11 - MATERIAL AND APPLICATION FACTOR

	Material and application factor (k_2)	
Consequence of failure classification*	**Basis for assignment of structural properties**	
	From in-grade verification **	**All other methods**
Normal	1.0	1.0
High	0.9	0.7

* Normal consequence of failure can be interpreted as that associated with housing construction, secondary framing in commercial or industrial scale structures and primary elements in farm buildings. High consequences of failure can be interpreted as that associated with primary structural elements in commercial or industrial scale structures, bridges and similar.

** In-grade verification involves the testing of the mechanical properties of a sample of timber to obtain test results confirming that these properties are in line with the grade of timber being assumed.

B3 DESIGN OF BASIC STRUCTURAL MEMBERS

B3.1 General

This Section shall be applied in conjunction with the clauses of Section B2. This Section applies to the design of basic structural members such as columns, beams and ties. In particular many of the design parameters given refer to members of rectangular cross-section, for which the notation used is shown in Figure B3.1.

Note: In beam design for domestic construction deflection considerations will usually govern member sizes. Deflections are not normally considered for timber bridges.

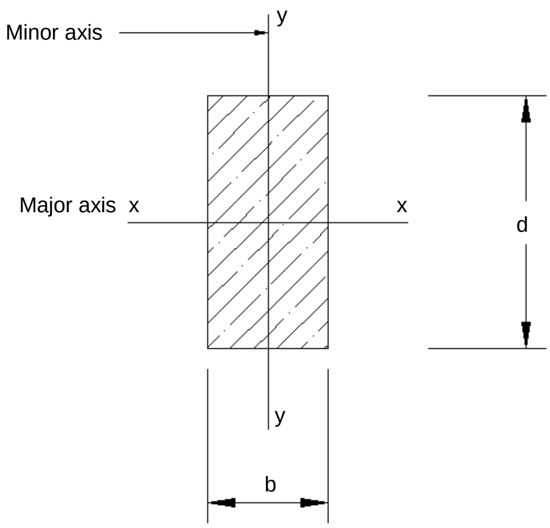


Figure B Error! No text of specified style in document.**.1- NOTATION FOR A RECTANGULAR CROSS-SECTION**

B3.2 Beam Design

B3.2.1 Maximum Stresses

Calculated values for the stresses in a beam shall not exceed the permissible stresses in Bending F_b, in shear F_s, in compression parallel to the grain F_c and in compression perpendicular to the grain F_p, determined in accordance with Clause B2.4 for sawn and laminated timber and for poles. Due regard shall be paid to the beam's effective span and lateral stability, and to an acceptable deflection. (See also Clause B3.5 for combined bending and axial loading).

When calculating a shear force in a beam, loads lying within a distance of the support of 1.5 times the depth of the beam from the inside face of the support may be disregarded, except in the design of notched beams.

For unnotched beams, the permissible stress shall be calculated by the following equations:

(a) In bending

$$F_b = k_1\, k_2\, k_4\, k_5\, k_6\, k_8\, k_{11}\, k_{12}\, F'_b \qquad \text{(B3.1)}$$

where the factor k_{12} is the stability factor defined in Equation B3.8 (see Clause B3.2.4).

(b) In shear

$$F_s = k_1\, k_2\, k_4\, k_5\, k_6\, F'_s \qquad \text{(B3.2)}$$

(c) In compression perpendicular to the grain
$F_p = k_1\ k_2\ k_4\ k_5\ k_6\ F'_p$ (B3.3)

For beams in grid systems the load-sharing factor k_8 is replaced by k_9.

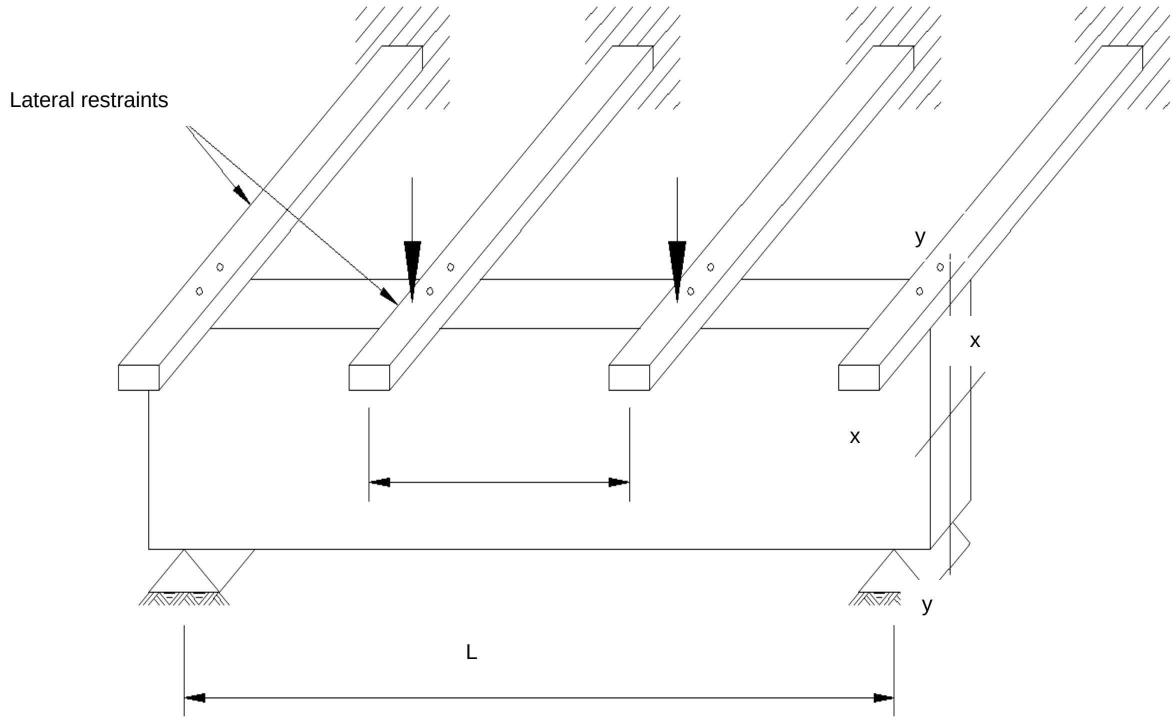


Figure B3.2 - NOTATION FOR BEAM RESTRAINTS

B3.2.2 Effective Span

The effective span of flexural members shall be taken as the distance between the centres of areas of bearing.

B3.2.3 Slenderness Coefficient for Lateral Buckling

B3.2.3.1 General

For solid beams of rectangular cross-section, the simple approximations given in Clause B3.2.3.2 may be used.

For sold or hollow beams of round or near round cross-section, lateral buckling need not be considered.

B3.2.3.2 Beams of Rectangular Cross-Section

For beams of rectangular cross-section, the slenderness coefficients may be taken as follows:

(a) Beams that only bend about their major axis.
For discrete restraint systems that effectively restrain the compression flange of the beam at points Lay apart, the slenderness coefficient denoted by S1 may be taken to be –

$S_1 = 1.25\ (L_{ay}\, d\ /\ b^2\ )^{1/2}$ (B3.4)

For restraint systems that are continuous along the compression flange of the beam, the slenderness coefficient may be taken to be –

$S_1 = 0.0$ (B3.5)

For restraint systems that are continuous along the tension flange of the beam, and in addition the load is applied to the tension flange, the slenderness coefficient may be taken to be –

$S_1 = 2.5\ d\ /\ b$ (B3.6)

(b) Beams that bend only about their minor axis.
For all cases, the slenderness coefficient, denoted by S_2 may be taken to be –

$S_2 = 0.0$ (B3.7)

(c) Beams that bend about both axes.
The design of such beams described in Clause B3.2.5, is based on an interaction of the two special cases for bending about single axes only, and hence no special definition of slenderness is required for this case.

B3.2.4 Stability Factor

The stability factor k_{12} for modification of the basic working stress in bending shall be given by –

(a) For $\rho S \leq 10$
$k_{12} = 1.0$ (B3.8(a))

(b) For $10 \leq \rho S \leq 20$
$k_{12} = 1.5 - 0.05\ \rho S$ (B3.8(b))

(c) For $\rho S > 20$
$k_{12} = 200\ /\ (\rho S)^2$ (B3.8(c))

where a conservative value of the material constant ρ is given in Table B3.1. The shape of the stability factor curve is illustrated in Figure B3.3.

Table B3.1 – MATERIAL CONSTANT FOR BEAMS, ρ

Stress Grade	Material Constant, ρ	
	Seasoned Timber	Unseasoned Timber
F34	1.23	1.32
F27	1.18	1.27
F22	1.13	1.22
F17	1.08	1.17
F14	1.04	1.14
F11	1.00	1.09
F8	0.95	1.05
F7	0.91	1.01
F5	0.88	0.97
F4	0.84	0.93
F3	0.80	0.90
F2	0.78	0.87

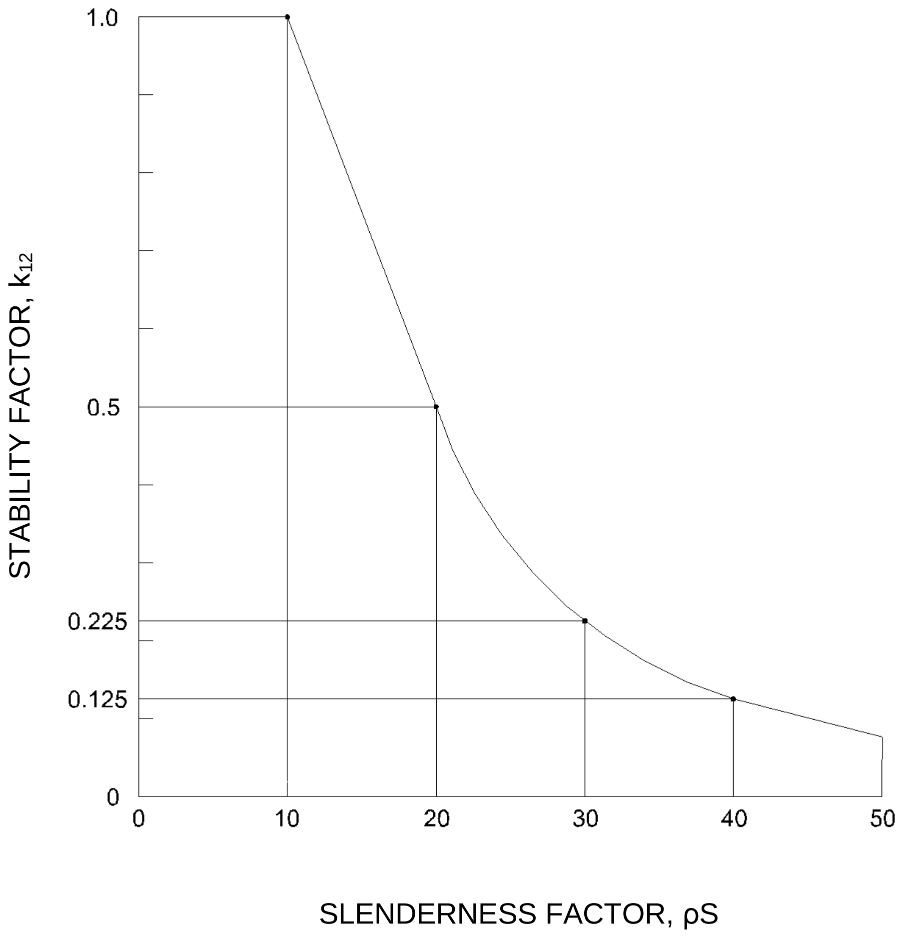


Figure B3.3 – EFFECT OF SLENDERNESS CO-EFFICIENT ON THE STABILITY FACTOR FOR BEAMS AND COLUMNS

For large beams, where a size factor $k_{11} < 1.0$ is used either for solid beams or the tension flanges of built up beams, the material constant ρ inserted in Equations B3.8 above may be replaced by ρ* where -

$$\rho^* = \rho\sqrt{k_{11}} \qquad \text{(B3.9)}$$

B3.2.5 Allowable Nominal Bending Stress

The following are design criteria for the allowable bending stress in a beam:

(a) Beam that is bent only about its major axis (the x-axis) -

$$f_{bx} / F_{bx} \leq 1 \qquad \text{(B3.10)}$$

(b) Beam that is bent only about its minor axis (the y-axis) -

$$f_{by} / F_{by} \leq 1 \qquad \text{(B3.11)}$$

(c) Beam that is bent about both major and minor axes -

$$(f_{bx} / F_{bx}) + (f_{by} / F_{by}) \leq 1 \qquad \text{(B3.12)}$$

where

f_{bx}, f_{by} = M_x/Z_x, M_y/Z_y and are calculated bending stresses about the major and minor axes respectively.

F_{bx}, F_{by} = permissible design values of f_{bx}, f_{by} if the beam were bent about only one axis.

B3.2.6 Strength of Notched Beams

For a rectangular beam of depth d, notched as shown in Figure B3.4, the nominal maximum bending stress $f_b = 6M/bd_n^2$ and nominal maximum shear stress $f_s = 3V/2bd_n$ calculated for the net section shall comply with the following interaction equation -

$$f_b + 4f_s \leq g_{40}F_{sj} \qquad \text{(B3.13)}$$

where g_{40} is computed as shown in Table B3.2 and F_{sj} is permissible shear stress for joint details from –

$$F_{sj} = k_1\, k_4\, k_5\, k_6\, k_{12}\, F'_{sj} \qquad \text{(B3.14)}$$

where the factors k_1 to k_{12} are given in Section B2

The stability factor k_{12} need not be considered in checking the fracture strength of notched beams, provided that the notice is not located within the middle third of the beam. Defects shall not be permitted within 150mm of the notch roots of critical beams, i.e. non-load-sharing beams.

If, according to the sign convention shown in Figure B3.4, f_b is negative, it may be taken as zero in the application of Equation B3.13. Similarly, if f_s is negative, it may also be taken as zero in the application of Equation B3.13.

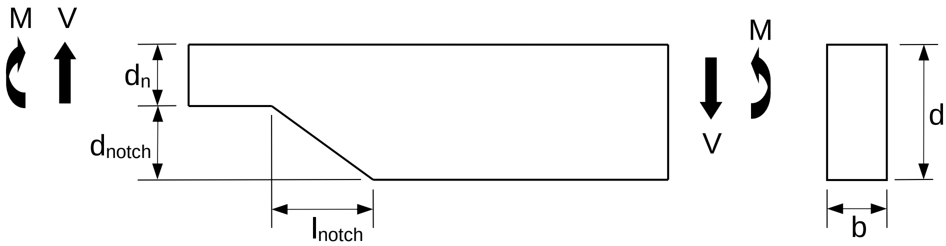


FIGURE B3.4 - NOTATION FOR NOTCH

Notch angle slope (see Figure B3.4)	g_{40}	
	d_{notch} > 0.1 d	d_{notch} < 0.1 d
$l_{notch} / d_{notch} = 0$	$9.0 / d^{0.45}$	$3.2 / d_{notch}^{0.45}$
$l_{notch} / d_{notch} = 2$	$9.0 / d^{0.33}$	$4.2 / d_{notch}^{0.33}$
$l_{notch} / d_{notch} = 4$	$9.0 / d^{0.24}$	$5.2 / d_{notch}^{0.24}$

TABLE B3.2 – COEFFICIENT g_{40} FOR SAWN NOTCH ON BEAM EDGE

B3.3 Column Design

B3.3.1 Maximum Stresses

The average compressive stress (f_c), calculated on the effective cross-sectional area A of a member concentrically loaded by an axial force P, shall not exceed the maximum permissible stress (F_c) in compression parallel to the grain as determined in accordance with Clause B2.4 for sawn and laminated timber and poles. (See also Clause B3.5.1 for combined bending and compression).

For unnotched columns, the permissible stress in compression shall be given by:

$$F_c = k_1\ k_2\ k_4\ k_5\ k_6\ k_8\ k_{12}\ F'_c \qquad \text{(B3.15)}$$

where the factors k_1 to k_8 are given in Section B2 and k_{12} is the stability factor defined by Equations B3.20(a), B3.20(b) and B3.20(c).

B3.3.2 Slenderness Coefficients for Lateral Buckling

B3.3.2.1 General

For the general case, and for several useful specific cases, equations for evaluating the slenderness coefficient are given in Appendix E of AS 1720.1-1997. For the case of solid columns of rectangular cross-section as shown in Figure B3.1, the simple approximations given below may be used.

B3.3.2.2 Columns of Rectangular Cross-Section

For columns of rectangular cross-section, the slenderness coefficients may be taken as follows:

(a) Columns that can bend only about their major axis.

For the case of discrete restraint systems, the slenderness coefficient, denoted by S_3, shall be taken to be the lesser of the following:

$S_3 = L_{ax} / d$ (B3.16(a))

and

$S_3 = g_{13} L / D$ (B3.16(b))

where

L_{ax} = the distance between points of effectively rigid restraint against lateral movement in the direction of the y-axis as shown in Figure B3.4(a)

g_{13} = the coefficient given in Table B3.2

For restraint systems that restrain movement in the direction of the y-axis, and are continuous along the length of the column, the slenderness coefficient may be taken to be:

$S_3 = 0.0$ (B3.17)

(b) Columns that can bend only about their minor axis.

For discrete restraint systems, the slenderness coefficient, denoted by S_4, may be taken to be the lesser of the following:

$S_4 = L_{ay} / b$ (B3.18(a))

and

$S_4 = g_{13} L / b$ (B3.18(b))

where

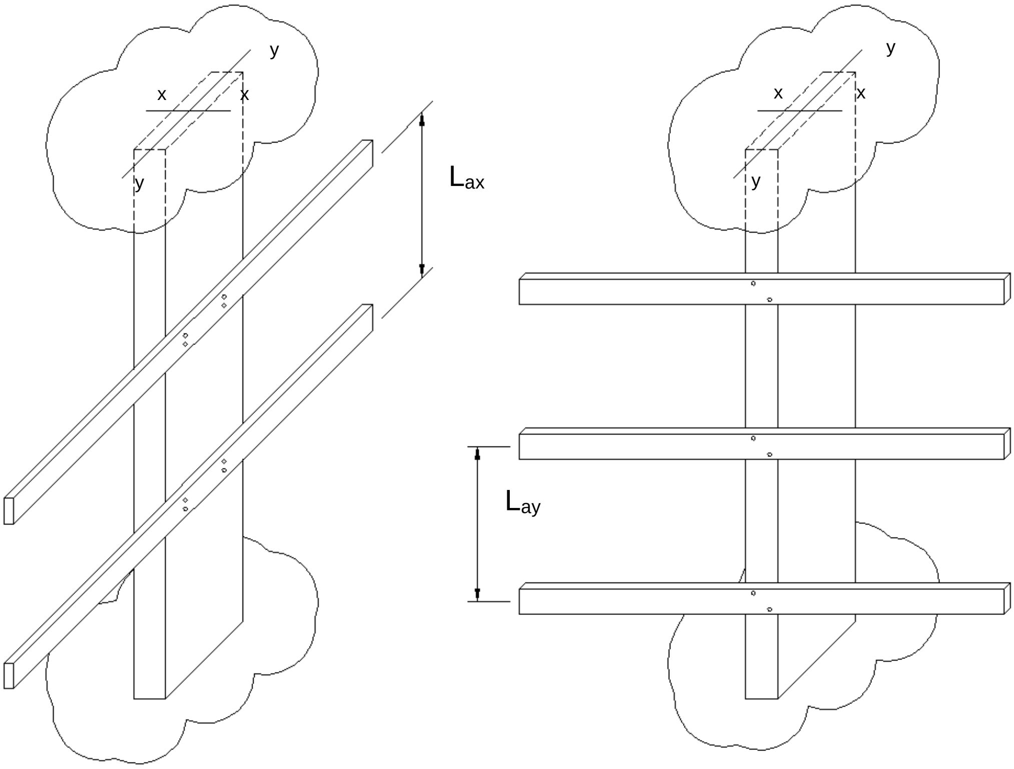
L_{ay} = the distance between points of effectively rigid restraints against lateral movement in the direction of the x-axis as shown in Figure B3.4(b)

g_{13} = the coefficient given in Table B3.2

For restraint systems that act continuously along one edge only and which restrain movement in the direction of the x-axis, the slenderness coefficient may be taken as:

$S_4 = 3.5\ d/b$ (B3.19)

(c) Columns that can bend about both axes.
The design of such column, described in Clause B3.5.1, is based on an interaction of the two special cases for bending about single axes only, and hence no special definition of slenderness is required for this case.



(a) For buckling about major axis (b) For buckling about minor axis

Figure B3.4 - NOTATION FOR COLUMN RESTRAINTS

Table B3.2 - EFFECTIVE LENGTH FACTOR g_{13}
FOR COLUMNS WITHOUT INTERMEDIATE LATERAL RESTRAINT

Condition of end restraint	Effective length factor (g_{13})
Flat ends	0.7
Restrained at both ends in position and direction	0.7
Each end held by two bolts (substantially restrained)	0.75
One end fixed in position and direction, the other restrained in position only	0.85
Restrained at both ends in position only	1.0
Restrained at one end in position and direction and at the other end partially restrained in direction but not in position	1.5
Restrained at one end in position and direction but not restrained in either position or direction at the other end	2.0

B3.3.3 Stability Factor

The stability factor k_{12} for modification of basic working stress in compression shall be given by:

(a) For $\rho S \le 10$

$$k_{12} = 1.0 \quad \text{(B3.20(a))}$$

(b) For $10 \le \rho S \le 20$

$$k_{12} = 1.5 - 0.05\,\rho S \quad \text{(B3.20(b))}$$

(c) For $\rho S \ge 20$

$$k_{12} = 200 / (\rho S)^2 \quad \text{(B3.20(c))}$$

where a conservative value of the material constant ρ is given in Table B3.3.

Table B3.3 - MATERIAL CONSTANT FOR COLUMNS, ρ

	Material Constant, ρ	
Stress Grade	**Seasoned Timber**	**Unseasoned Timber**
F34	1.26	1.44
F27	1.22	1.39
F22	1.18	1.35
F17	1.13	1.30
F14	1.10	1.27
F11	1.06	1.22
F8	1.01	1.18
F7	0.98	1.15
F5	0.95	1.11
F4	0.91	1.07
F3	0.88	1.04
F2	0.85	1.01

B3.3.4 Allowable Nominal Axial Stress

Allowable compression stress in a column is given by -

$$f_c \le F_{cx} \quad \text{(B3.21)}$$

$$f_c \le F_{cy} \quad \text{(B3.22)}$$

where

f_c = nominal compressive stress in column = P/A

F_{cx}, F_{cy} = permissible compressive stress for the member as a column able to buckle only about its major or minor axes respectively, (see Equation B3.13)

B3.3.5 Strength of Notched Columns

For a column, notched in the middle third, and with a stability factor $k_{12} < 0.5$, a check shall be made that the fracture strength is adequate.

The fracture strength may be considered to the adequate if the member, considered as beam, is capable of sustaining a nominal bendress stress $f_b=k_1F_c(1-2k_{12})$ at the notch root when a check is made in accordance with Clause B3.2.6.

B3.4 Tension Member Design

B3.4.1 Axial Stress in Tension Members

In an axially loaded tension member, the average tensile stress (f_t), calculated on the net area, shall not exceed the permissible stress in tension (F_t), determined in accordance with Clause B2.4 for sawn and laminated timber and poles. (See also Clause B3.5.2 for combined bending and tension).

For unnotched tension members, the permissible stress in tension shall be given -

$$F_t = k_1 k_2 k_4 k_5 k_6 k_{11} F'_t \quad \text{(B3.23)}$$

where the factors k_1 to k_{11} are given in Section B2.

B3.4.2 Slenderness Coefficient

The slenderness coefficient for a tension member shall be defined as for columns in accordance with Clause B3.3.2.

B3.5 Combined Bending and Axial Stresses

B3.5.1 Combined Bending and Compression

A rectangular member with cross-section as shown in Figure B3.1 subject to combined axial compression and bending about the x-axis only shall be proportioned so that -

$$(f_{bx} / F_{bx})^2 + (f_c / F_{cy}) \leq 1 \quad \text{(B3.24)}$$

$$(f_{bx} / F_{bx}) + (f_c / F_{cx}) \leq 1 \quad \text{(B3.25)}$$

where

f_{bx} = M_x/Z_x = nominal bending stress about the major axis

F_{bx} = permissible design values of f_{bx} (Equation B3.1)

f_c = P/A = nominal compression stress acting on column

F_{cx}, F_{cy} = permissible design value of the compression stress (f_c) if the member were used as a column that could buckle only about its major or minor axis respectively

Note: Equations B3.24 and B3.25 contain an allowance for the effect of bending moment amplification due to the axial load. For non-rectangular members, Equations B3.24 and B3.25 may be used in the absence of other information.

B3.5.2 Combined Bending and Tension

The nominal bending stress f_{bx} and axial stress f_t of a member subject to combined bending and axial tension shall be given by -

$$0.6\ f_{bx} + f_t \leq F_t \qquad \text{(B3.26)}$$

$$f_{bx} - f_t \leq F_{bx} \qquad \text{(B3.27)}$$

where F_t and F_{bx} are the permissible tension and bending stresses for the member used as a tie or beam respectively (see Equations B3.1 and B3.23).

B4 CONNECTIONS

B4.1 General

B4.1.1 Scope of Section

This Section applies to joints in solid timber fabricated with bolted fasteners only.

B4.1.2 Joint Groups

For the purpose of joint design, timber species have been classified into six joint groups: J1, J2, J3, J4, J5 and J6 for unseasoned timber and JD1, JD2, JD3, JD4, JD5 and JD6 for seasoned timber. The joint group classifications for specific timbers are shown in Tables B2.1 and B2.2.

Where joints comprise more than one species of timber, the design load to be used in the absence of other information is that appropriate to the weakest species in the joint.

B4.1.3 Timber Grade

No allowance for grade of timber has been made in design data for fasteners. Design loads for joints have been based on the assumption that there are no loose knots, severe sloping grain, gum veins, gum or rot pockets, lyctus-susceptible sapwood, corewood, holes or splits near any fastener. Accordingly, all of these defects except for corewood shall be avoided at fastener locations. Corewood shall only be permitted at fastener locations if the design joint strength is taken to be that of timber in a joint group one lower than the normal value for the species used.

B4.1.4 Tendency to Split

Special precautions shall be specified in the use of timber that has a tendency to split to an extent that may be detrimental to connector strength. In the absence of other guidance, the criterion for tendency to split shall be defined by:

$$\alpha = \varepsilon^2 / y$$

where

ε = tangential shrinkage, in percent.

y = tangential cleavage strength of unseasoned timber, in Newtons per millimetre (N/mm), as measured by BS 373 or ASTM D143.

Species for which $\alpha > 0.8$ often have a high tendency to split, particularly in exposed locations; species for which $\alpha < 0.55$ may be considered to have a negligible tendency to split.

B4.2 Not Used

B4.3 Not Used

B4.4 Bolted Joints

B4.4.1 General

The basic working loads given in Clauses B4.4.2.1 and B4.4.2.2 are applicable to steel bolts as specified in AS 1111, when fitted into pre-bored holes of diameter approximately 10 percent greater than the bolt diameter and when fitted with washers as given in Clause B4.4.2.6.

B4.4.2 Lateral Loads

B4.4.2.1 Basic Working Load Parallel to the Grain

The basic working load Q'_a for a single bolt bearing parallel to the grain and acting in single shear is given for a selection of bolt diameters and effective timber thicknesses in Tables B4.1(B) and B4.1(C).

B4.4.2.2 Basic Working Load Perpendicular to the Grain

The basic working load Q'_p for a single bolt bearing perpendicular to the grain and acting in single shear is given for a selection of bolt diameters and effective timber thicknesses in Tables B4.2(B) and B4.2(C).

B4.4.2.3 Basic Working Load for a Bolted Joint System

The basic working load Q'_s is derived as follows:

(a) For systems loaded parallel to the grain, $Q'_s = Q'_{sa}$,
where Q'_{sa} is the system capacity given in Table 4.1(A).

(b) For systems loaded perpendicular to the grain, $Q'_s = Q'_{sp}$,
where Q'_{sp} is the system capacity given in Table 4.2(A).

(c) For systems loaded at an angle to the grain, the system capacity is given by use of Hankinson's formula as follows:

$$Q'_s = \{Q'_{sa}\, Q'_{sp}\} / \{Q'_{sa} \sin^2 \theta + Q'_{sp} \cos^2 \theta\} \qquad \text{(B4.1)}$$

where

θ = the angle between the load direction and the grain direction

Hankinson's formula is conveniently evaluated by means of the nomograms given in Figures B4.1 to B4.3.

B4.4.2.4 Permissible Loads

The permissible load Q_S of a laterally loaded bolt system shall be taken to be given by -

$$Q_S = k_1 k_{16} Q'_S \quad \text{(B4.2)}$$

where

k_1 = factor for duration of load given in Table B2.5

k_{16} = 1.2 for bolts that transfer load through metal side plates of adequate strength, and bolts are a close fit to the holes in the plates provided that b/D > 5 for loads acting parallel to the grain and b/D>10 for loads acting perpendicular to the grain (where b denotes the effective timber thickness and D is the bolt diameter)

= 1.0 otherwise

Q'_S = the basic working load as derived in Clause B4.4.2.3

B4.4.2.5 Spacings, Edge and End Distances

Spacings, edge and end distances shall comply with the following requirements:

(a) Loads parallel to grain

The basic working loads given in Tables B4.1(A), B4.1(B) and B4.1(C) apply to joints in which the edge, end and between-fastener spacings are not less than those shown in Figure B4.4(a). The distance a indicated in the figure shall be at least (n – 2)D with a minimum of 2.5D, where n is the total number of bolts in the joint and D is the diameter of the bolt.

Similarly, the required end distance ℓ_{par} shall be at least 8D in tension joints in unseasoned timber, 7D in tension joints in seasoned timber and 5D in compression joints and in joints subject to bending moment for both moisture conditions. However, lesser end distances may be used in tension joints provided that the basic load is reduced in proportion to the reduction in end distance.

Nevertheless, in no case shall the end distance for tension joints be less than 6D for unseasoned timber and 5D for seasoned timber.

(b) Loads perpendicular to grain

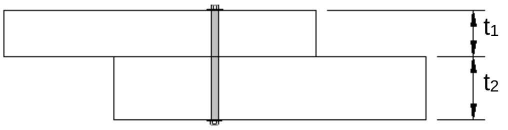
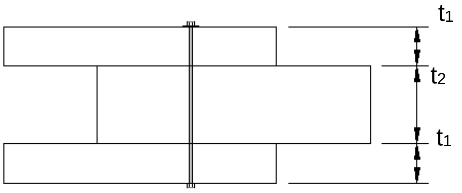
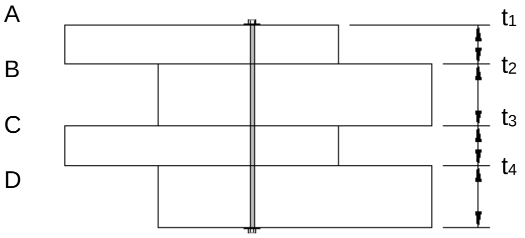
The minimum edge, end and between-fastener spacings shall not be less than those shown in Figure B4.4(b). The distance a shall be at least 2.5D for a b/D ratio of 2, and it shall be increased proportionately so that it is at least 5D for a b/D ratio of 6 or more, where b is the thickness of the member loaded perpendicular to the grain.

(c) Loads acting at an angle to the grain

For loads acting at an angle 0° to 30° to the grain, the spacings, edge and end distances may be taken as for loads parallel to the grain. For loads acting at an angle of 30° to 90° to the grain, the spacings, edge and end distances may be taken as for loads acting perpendicular to the grain.

Table B4.1 - BASIC WORKING LOADS FOR SINGLE BOLTS PARALLEL TO GRAIN
(COMPRISING Tables B4.1(A), B4.1(B) AND B4.1(C))

Table B4.1(A) - SYSTEM CAPACITY

<table>
<tr><th>Type of Joint</th><th>Effective timber thickness</th><th>System Capacity Q'_{sa}</th></tr>
<tr><td>1. Two member
</td><td>Smaller of t_1 and t_2</td><td>Q'_a</td></tr>
<tr><td>2. Three member
</td><td>Smaller of t_2 and $2t_1$</td><td>2 Q'_a</td></tr>
<tr><td>3. Multiple member
</td><td>(i) Between A and B - smaller of t_1 and t_2
(ii) Between B and C - smaller of t_2 and t_3
(iii) etc.</td><td>(i) Q'_a
(ii) Q'_a
(iii) etc

Q'_{sa} = sum of basic loads (i), (ii), etc.</td></tr>
</table>

Table B4.1(B) - IN UNSEASONED TIMBER : PARALLEL TO GRAIN

Joint group	Effective timber thickness mm	Basic working load Q'a , N								
		Bolt diameter								
		M6	M8	M10	M12	M16	M20	M24	M30	M36
J1	25	980	1650	2100	2500	3300	4100	5000	6200	7400
	38	980	1740	2700	3800	5000	6300	7500	9400	11300
	50	980	1740	2700	3900	6600	8300	9900	12400	14900
	75	980	1740	2700	3900	7000	10900	14900	18600	22000
	100	980	1740	2700	3900	7000	10900	15700	25000	30000
	150	980	1740	2700	3900	7000	10900	15700	25000	35000
	200	980	1740	2700	3900	7000	10900	15700	25000	35000
J2	25	820	1300	1630	1950	2600	3300	3900	4900	5900
	38	820	1460	2300	3000	4000	4900	5900	7400	8900
	50	820	1460	2300	3300	5200	6500	7800	9800	11700
	75	820	1460	2300	3300	5800	9100	11700	14600	17600
	100	820	1460	2300	3300	5800	9100	13100	19500	23000
	150	820	1460	2300	3300	5800	9100	13100	20000	29000
	200	820	1460	2300	3300	5800	9100	13100	20000	29000
J3	25	760	1050	1310	1580	2100	2600	3200	3900	4700
	38	760	1340	2000	2400	3200	4000	4800	6000	7200
	50	760	1340	2100	3000	4200	5300	6300	7900	9500
	75	760	1340	2100	3000	5400	7900	9500	11800	14200
	100	760	1340	2100	3000	5400	8400	12100	15800	18900
	150	760	1340	2100	3000	5400	8400	12100	18900	27000
	200	760	1340	2100	3000	5400	8400	12100	18900	27000
J4	25	600	830	1040	1250	1660	2100	2500	3100	3700
	38	600	1060	1580	1890	2500	3200	3800	4700	5700
	50	600	1060	1660	2400	3300	4200	5000	6200	7500
	75	600	1060	1660	2400	4200	6200	7500	9300	11200
	100	600	1060	1660	2400	4200	6600	9600	12500	14900
	150	600	1060	1660	2400	4200	6600	9600	14900	22000
	200	600	1060	1660	2400	4200	6600	9600	14900	22000
J5	25	500	660	830	990	1320	1650	1980	2500	3000
	38	520	930	1250	1500	2000	2500	3000	3800	4500
	50	520	930	1450	1980	2600	3300	4000	5000	5900
	75	520	930	1450	2100	3700	5000	5900	7400	8900
	100	520	930	1450	2100	3700	5800	7900	9900	11900
	150	520	930	1450	2100	3700	5800	8400	13100	17800
	200	520	930	1450	2100	3700	5800	8400	13100	18800
J6	25	400	530	660	800	1060	1330	1590	1990	2400
	38	460	810	1010	1210	1610	2000	2400	3000	3600
	50	460	810	1270	1590	2100	2700	3200	4000	4800
	75	460	810	1270	1830	3200	4000	4800	6000	7200
	100	460	810	1270	1830	3300	5100	6400	8000	9500
	150	460	810	1270	1830	3300	5100	7300	11400	14300
	200	460	810	1270	1830	3300	5100	7300	11400	16500

Table B4.1(C) - IN SEASONED TIMBER : PARALLEL TO GRAIN

Joint group	Effective timber thickness mm	Basic working load Q'_a, N								
		Bolt diameter								
		M6	M8	M10	M12	M16	M20	M24	M30	M36
JD1	20	1200	1640	2100	2500	3300	4100	4900	6200	7400
	30	1200	2200	3100	3700	4900	6200	7400	9200	11100
	40	1200	2200	3400	4900	6600	8200	9800	12300	14800
	50	1200	2200	3400	4900	8200	10300	12300	15400	18500
	70	1200	2200	3400	4900	8700	13500	17200	22000	26000
	85	1200	2200	3400	4900	8700	13500	19500	26000	31400
	100	1200	2200	3400	4900	8700	13500	19500	30000	37000
JD2	20	990	1320	1650	1980	2600	3300	4000	5000	5900
	30	1040	1850	2500	3000	4000	5000	5900	7400	8900
	40	1040	1850	2900	4000	5300	6600	7900	9900	11900
	50	1040	1850	2900	4200	6600	8300	9900	12400	14900
	70	1040	1850	2900	4200	7400	11600	13900	17300	21000
	85	1040	1850	2900	4200	7400	11600	16600	21000	25000
	100	1040	1850	2900	4200	7400	11600	16600	25000	30000
JD3	20	780	1040	1300	1560	2100	2600	3100	3900	4700
	30	940	1560	1950	2300	3100	3900	4700	5900	7000
	40	940	1660	2600	3100	4200	5200	6200	7800	9400
	50	940	1660	2600	3700	5200	6500	7800	9800	11700
	70	940	1660	2600	3700	6700	9100	10900	13700	16400
	85	940	1660	2600	3700	6700	10400	13300	16600	19900
	100	940	1660	2600	3700	6700	10400	15000	19500	23000
JD4	20	630	840	1050	1260	1680	2100	2500	3200	3800
	30	760	1260	1580	1890	2500	3200	3800	4700	5700
	40	760	1340	2100	2500	3400	4200	5000	6300	7600
	50	760	1340	2100	3000	4200	5300	6300	7900	9500
	70	760	1340	2100	3000	5400	7400	8800	11000	13200
	85	760	1340	2100	3000	5400	8400	10700	13400	16100
	100	760	1340	2100	3000	5400	8400	12100	15800	18900
JD5	20	500	660	830	1000	1330	1660	1990	2500	3000
	30	660	1000	1250	1490	1990	2500	3000	3700	4500
	40	660	1170	1660	1990	2700	3300	4000	5000	6000
	50	660	1170	1830	2500	3300	4200	5000	6200	7500
	70	660	1170	1830	2600	4600	5800	7000	8700	10500
	85	660	1170	1830	2600	4700	7100	8500	10600	12700
	100	660	1170	1830	2600	4700	7300	10000	12500	14900
JD6	20	400	530	660	790	1060	1320	1580	1980	2400
	30	570	790	990	1190	1580	1980	2400	3000	3600
	40	570	1010	1320	1580	2100	2600	3200	4000	4800
	50	570	1010	1580	1980	2600	3300	4000	5000	5900
	70	570	1010	1580	2300	3700	4600	5500	6900	8300
	85	570	1010	1580	2300	4100	5600	6700	8400	10100
	100	570	1010	1580	2300	4100	6300	7900	9900	11900

Table B4.2 - BASIC WORKING LOADS FOR SINGLE BOLTS PERPENDICULAR TO THE GRAIN
(COMPRISING Tables B4.2(A), B4.2(B) AND B4.2(C))

Table B4.2(A) - SYSTEM CAPACITY

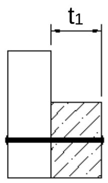
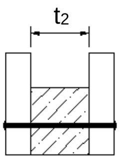
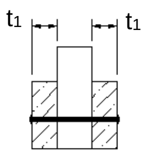
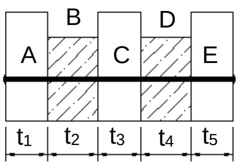
Type of Joint	Effective timber thickness	System Capacity Q'_{sp}
1. Two member 	$2t_1$	Q'_p
2. Three member, Type A 	t_2	$2\ Q'_p$
3. Three member, Type B 	$2t_1$	$2\ Q'_p$
4. Multiple member 	(i) Between A and B - t_2 (ii) Between B and C - t_2 (iii) Between C and D - t_4 (iv) etc.	(i) Q'_p (ii) Q'_p (iii) Q'_p (iv) etc Q'_{sp} = sum of basic loads (i), (ii), (iii), etc.
Note: At each interface, the strength of the bolted joint with respect to the member aligned parallel to the direction of the stress must be checked according to Table B4.1.		

Table B4.2(B) - IN UNSEASONED TIMBER : PERPENDICULAR TO GRAIN

Joint group	Effective timber thickness mm	Basic working load Q'_p , N								
		Bolt diameter								
		M6	M8	M10	M12	M16	M20	M24	M30	M36
J1	25	500	660	830	990	1320	1650	1980	2500	3000
	38	750	1000	1250	1500	2000	2500	3000	3800	4500
	50	970	1320	1650	1980	2600	3300	4000	5000	5900
	75	970	1490	2100	2700	4000	5000	5900	7400	8900
	100	970	1490	2100	2700	4200	5900	7800	9900	11900
	150	970	1490	2100	2700	4200	5900	7800	10800	14300
	200	970	1490	2100	2700	4200	5900	7800	10800	14300
J2	25	390	520	650	780	1040	1330	1560	1950	2300
	38	590	790	990	1190	1580	1980	2400	3000	3600
	50	780	1040	1300	1560	2100	2600	3100	3900	4700
	75	920	1410	1950	2300	3100	3900	4700	5900	7000
	100	920	1410	1970	2600	4000	5200	6200	7800	9400
	150	920	1410	1970	2600	4000	5600	7300	10300	13500
	200	920	1410	1970	2600	4000	5600	7300	10300	13500
J3	25	250	330	410	500	660	830	990	1240	1490
	38	380	500	630	750	1000	1250	1500	1880	2300
	50	500	660	830	990	1320	1650	1980	2500	3000
	75	730	990	1240	1490	1980	2500	3000	3700	4500
	100	730	1120	1570	1980	2600	3300	4000	5000	5900
	150	730	1120	1570	2100	3200	4400	5800	7400	8900
	200	730	1120	1570	2100	3200	4400	5800	8100	10700
J4	25	157	210	260	310	420	520	630	790	940
	38	240	320	400	480	640	800	960	1200	1440
	50	310	420	520	630	840	1050	1260	1570	1890
	75	470	630	790	950	1260	1580	1890	2400	2800
	100	520	810	1050	1260	1680	2100	2500	3100	3800
	150	520	810	1130	1480	2300	3200	3800	4700	5700
	200	520	810	1130	1480	2300	3200	4200	5900	7600
J5	25	105	140	175	210	280	350	420	530	630
	38	159	210	270	320	430	530	640	800	960
	50	210	280	350	420	560	700	840	1050	1260
	75	320	420	530	630	840	1050	1260	1580	1890
	100	390	560	700	840	1120	1400	1680	2100	2500
	150	390	600	840	1110	1680	2100	2500	3200	3800
	200	390	600	840	1110	1700	2400	3100	4200	5000
J6	25	52	70	87	105	140	175	210	260	320
	38	79	106	133	159	210	270	320	400	480
	50	105	140	175	210	280	350	420	530	630
	75	157	210	260	320	420	530	630	790	950
	100	195	280	350	420	560	700	840	1050	1260
	150	195	300	420	550	840	1050	1260	1580	1890
	200	195	300	420	550	850	1190	1560	2100	2500

Table B4.2(C) - IN SEASONED TIMBER : PERPENDICULAR TO GRAIN

Joint group	Effective timber thickness mm	Basic working load Q'_p, N								
		Bolt diameter								
		M6	M8	M10	M12	M16	M20	M24	M30	M36
JD1	20	520	700	870	1040	1390	1740	2100	2600	3100
	30	780	1040	1310	1570	2100	2600	3100	3900	4700
	40	1040	1390	1740	2100	2800	3500	4200	5200	6300
	50	1280	1740	2200	2600	3500	4400	5200	6500	7800
	70	1280	1970	2800	3600	4900	6100	7300	9100	11000
	85	1280	1970	2800	3600	5600	7400	8900	11100	13300
	100	1280	1970	2800	3600	5600	7800	10200	13100	15700
JD2	20	400	540	670	800	1070	1340	1610	2000	2400
	30	600	800	1010	1210	1610	2000	2400	3000	3600
	40	800	1070	1340	1610	2100	2700	3200	4000	4800
	50	1010	1340	1680	2000	3700	3400	4000	5000	6000
	70	1180	1820	2300	2800	3800	4700	5600	7000	8400
	85	1180	1820	2500	3300	4600	5700	6800	8500	10300
	100	1180	1820	2500	3300	5100	6700	8000	10100	12100
JD3	20	300	400	500	600	800	1000	1200	1500	1800
	30	450	600	750	900	1200	1500	1800	2300	2700
	40	600	800	1000	1200	1600	2000	2400	3000	3600
	50	750	1000	1250	1500	2000	2800	3000	3800	4500
	70	1050	1400	1750	2100	2800	3500	4200	5300	6300
	85	1100	1700	2100	2600	3400	4300	5100	6400	7700
	100	1100	1700	2400	3000	4000	5000	6000	7500	9000
JD4	20	220	300	370	440	590	740	890	1110	1330
	30	330	440	560	670	890	1110	1330	1670	2000
	40	440	590	740	890	1180	1480	1780	2200	2700
	50	560	740	930	1110	1480	1850	2200	2800	3300
	70	780	1040	1300	1550	2100	2600	3100	3900	4700
	85	920	1260	1570	1890	2500	3100	3800	4700	5700
	100	920	1420	1850	2200	3000	3700	4400	5600	6700
JD5	20	156	210	260	310	420	520	620	780	940
	30	230	310	390	470	620	780	940	1170	1400
	40	310	420	520	620	830	1040	1300	1560	1870
	50	390	520	650	780	1040	1300	1560	1950	2300
	70	550	730	910	1090	1460	1820	2200	2700	3300
	85	660	880	1100	1330	1770	2200	2700	3300	4000
	100	780	1040	1300	1560	2100	2600	3100	3900	4700
JD6	20	108	144	180	220	290	360	430	540	650
	30	162	220	270	320	430	540	650	810	970
	40	220	290	360	430	580	720	860	1080	1300
	50	270	360	450	540	720	900	1080	1350	1620
	70	380	500	630	760	1010	1260	1510	1890	2300
	85	460	610	770	920	1220	1530	1840	2300	2800
	100	540	720	900	1080	1440	1800	2200	2700	3200

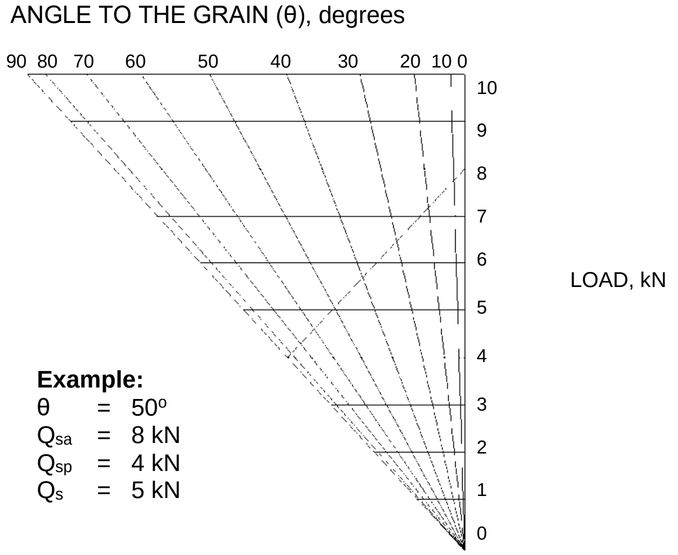


Figure B4.1 - NOMOGRAM FOR HANKINSON'S FORMULA, RANGE I

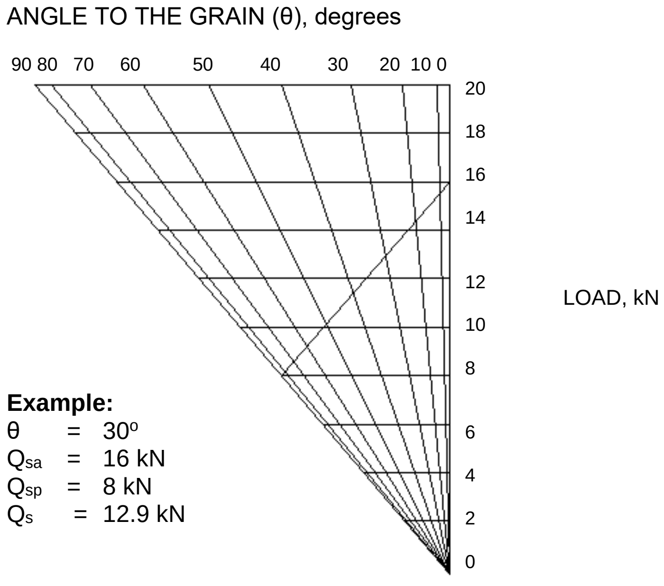


Figure B4.2 - NOMOGRAM FOR HANKINSON'S FORMULA, RANGE II

ANGLE TO THE GRAIN (θ), degrees

LOAD, kN

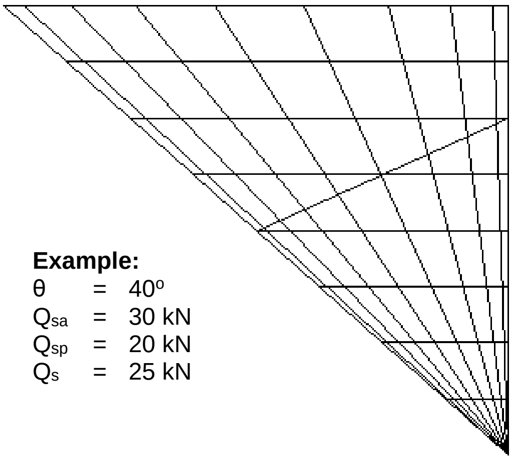


Figure B4.3 - NOMOGRAM FOR HANKINSON'S FORMULA, RANGE III

B4.4.2.6 Washers

In all timber-to-timber bolted structural joints, every bolt shall be provided with a washer at each end, of a size not less than in Table B4.3. If smaller washers are used then the basic working load given in Clause B4.4.2 shall be reduced in proportion to the dimension of the washer diameter or side length.

Table B4.3 - MINIMUM REQUIRED SIZE OF WASHERS FOR STRUCTURAL BOLTED JOINTS

Bolt Size	Washer size, mm		
	Thickness	**Minimum diameter for round washers**	**Minimum side length for square washers**
M6	1.6	30	25
M8	2.0	36	32
M10	2.5	45	40
M12	3.0	55	50
M16	4.0	65	57
M20	5.0	72	65
>M20	6.0	85	75

B4.4.3 Axial Loads

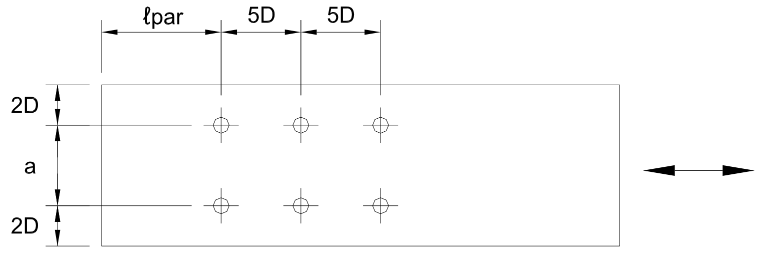
Where bolts are loaded axially, the basic working load of the bolt shall be taken as the lesser of the axial strength of the bolt and the bearing strength of the timber under the washer when loaded perpendicular to the grain. The design axial strength of bolts and the effective diameter for use in computing the bearing pressure on the timber are given in Table B4.4.

Table B4.4 - DESIGN PARAMETERS FOR BOLTS UNDER AXIAL LOAD

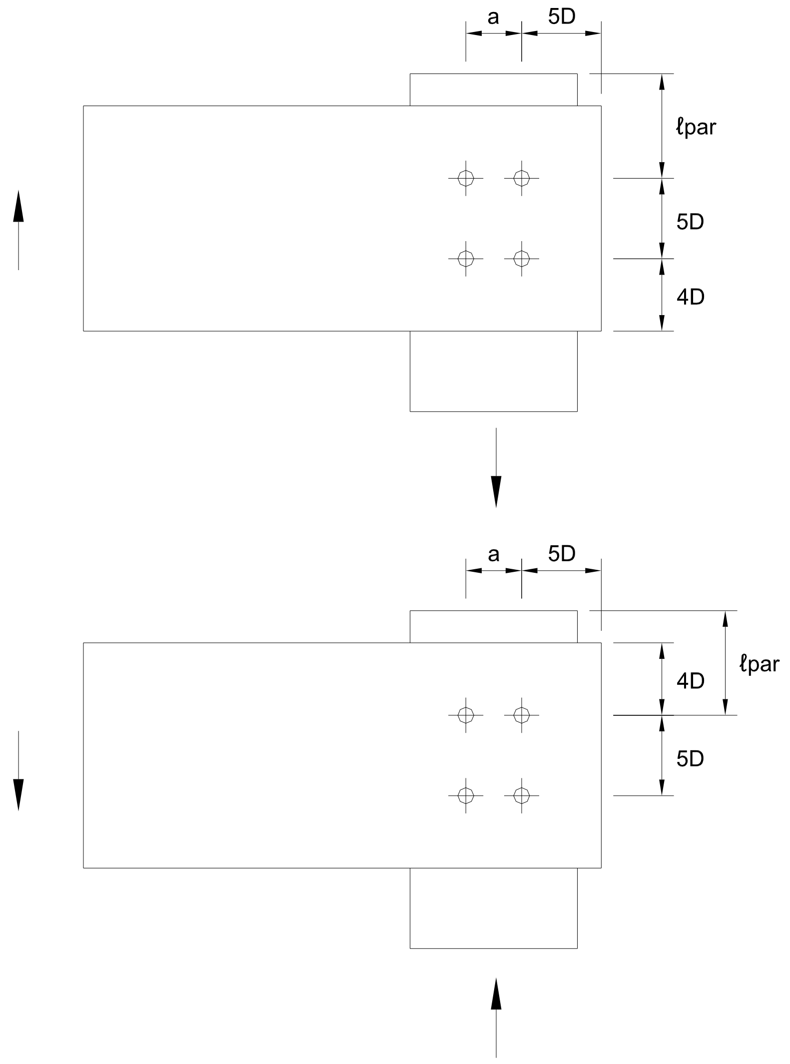
Bolt Size	Axial strength of bolt* (kN)	Effective diameter of a standard washer in bearing# (mm)
M6	4.0	16
M8	7.5	21
M10	11.5	27
M12	17.0	35
M16	32.0	44
M20	50.0	50
M24	72.0	60
M30	115.0	69
M36	165.0	78

* Bolts to be of grade 4.6, AS 1111.

Standard washers are washers having the minimum dimensions shown in Table B4.3. The effective diameter is less than the actual diameter because it includes an allowance for bending of the washer.



(a) Load applied parallel to grain



(b) Load applied perpendicular to grain

Figure B4.4 – SPACING, EDGE AND END DISTANCES FOR BOLTED JOINTS

B5 ROUND TIMBERS

B5.1 General

Whether naturally round timbers are used as simple structural members, i.e. as poles or piles or as elements of a composite structure, the design procedures shall be similar to those given in Section B3, subject to the provisions of Clauses B5.2, B5.3 and B5.4.

B5.2 Basic Working Stresses and Stiffness

The basic working stresses and stiffness for untrimmed logs, poles or piles conforming in quality to the requirements of AS 2209 shall be those given in Tables B2.3 and B2.4. For any particular species, the appropriate stress grade is derived from its strength groups as given in Table B5.1.

Table B5.1 - CORRESPONDENCE BETWEEN STRENGTH GROUP AND STRESS GRADE FOR ROUND TIMBERS GRADED TO AS 2209

Strength group	Stress grade
S1	F34
S2	F27
S3	F22
S4	F17
S5	F14
S6	F11
S7	F8

Note: The equivalence expressed in Table B5.1 is based on the assumption that all poles or logs are cut from mature trees. Factors for immaturity are given in Clause B5.4.1.

B5.3 Design

B5.3.1 Permissible Stresses

To obtain permissible stresses for naturally round timbers, the basic working stresses shall be modified by such factors given in Clause B2.5 as are applicable to the service conditions. In addition, the modification factors specified in Clause B5.4 shall also be applied where appropriate.

B5.3.2 Deflections

Deflection calculations shall take into account the modification factors in Clause B2.5.1.2.

B5.4 Additional Modification Factors

B5.4.1 Factor for Immaturity

For poles having mid length diameters less than 250 mm, due allowance must be made for the properties of immature timber. For eucalyptus species and radiata pine, this may be done through multiplication of basic stresses and modulus of elasticity by the factors k_{20} and j_9 respectively given in Table B5.2.

Note: For species other than eucalyptus or radiata pine, conservative assumptions should be used in design unless special investigations have been undertaken to derive accurate values.

Table B5.2(A) - IMMATURITY FACTOR k_{20} FOR STRESSES

Species	Factor k_{20} for stresses							
	D = 75	D = 100	D = 125	D = 150	D = 175	D = 200	D = 225	D = 250
Eucalyptus species	0.80	0.90	1.00	1.00	1.00	1.00	1.00	1.00
Radiata pine	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.00

D = pole diameter at mid-length, mm.

Table B5.2(B) - IMMATURITY FACTOR j_9 FOR STIFFNESS

Species	Factor k_{20} for stresses							
	D = 75	D = 100	D = 125	D = 150	D = 175	D = 200	D = 225	D = 250
Eucalyptus species	0.80	0.90	1.00	1.00	1.00	1.00	1.00	1.00
Radiata pine	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.00

D = pole diameter at mid-length, mm.

B5.4.2 Shaving Factor

For timber members in natural pole form, the basic working stresses shall be reduced if the poles have been shaved. For poles of eucalyptus species and radiata pine that have been shaved to a smooth cylindrical form, the shaving factor k_{21} shall be taken as specified in Table B5.3. In addition, it shall be assumed that the effect of shaving will be to reduce the modulus of elasticity by 5 percent.

Table B5.3 - SHAVING FACTOR k_{21}

Stress	**Factor k_{21}**	
	Eucalyptus Species	**Radiata Pine**
Bending	0.85	0.75
Compression parallel to grain	0.95	0.90
Compression perpendicular to grain and shear	1.00	1.00
Tension	0.85	0.75

B5.5 Design Details

B5.5.1 Effective Pole Cross-Section

The effective diameter of a cross-section between two points of lateral restraint shall be taken as the mean of the diameters at the points of lateral restraint.

B5.5.2 Effective Cross-Section of Untreated Timber

Unless subjected to adequate preservative treatment in accordance with an approved Standard, the sapwood of all timbers shall be disregarded in assessing the effective structural cross-section of poles at or above the ground-line where exposed to the weather or when used as piles above permanent water level.

B5.5.3 Moisture Content of Timbers in Ground Contact

Irrespective of whether poles are used in the unseasoned, partially seasoned or fully seasoned condition, it shall be assumed that all parts of poles within 1m of a ground-line contact are, for design purposes, in the unseasoned condition.

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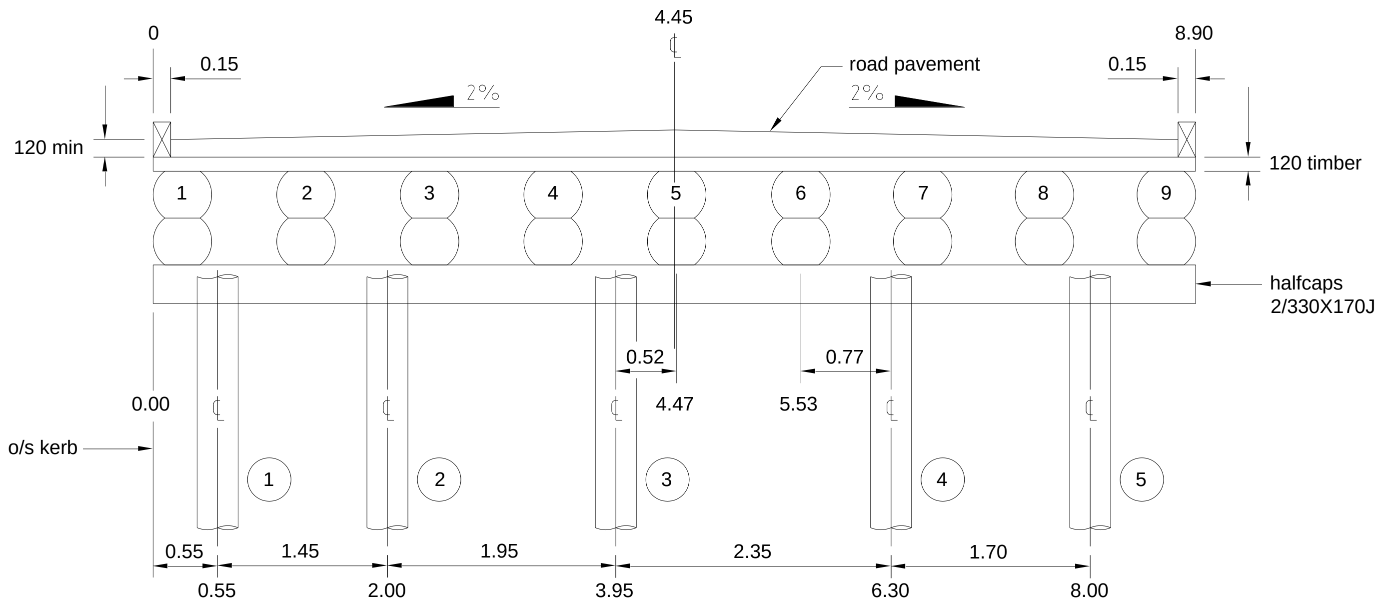
APPENDIX C

Example of Halfcap Analysis

There is some concern that the halfcaps on Pier 1 could be overloaded in shear and/or bending due to the large span and smaller than normal size of halfcaps.

- Pier 1 Geometry:

  Width overall = 8.90m
  Width between kerbs = 8.60m
  Number of stringers = 9
  Number of piles = 5



Pier 1 Geometry

By inspection of the geometry above the section of halfcap spanning between piles 3 and 4 (span of 2.35 m) which supports two stringers, may be overstressed in shear or bending.

To calculate the load rating of the halfcap the loads applied to this halfcap from the stringers/corbels first must be derived.

These loads shall be obtained from TIMBAR. A standard TIMBAR analysis determines the maximum load on each stringer by moving the vehicle transversely across the bridge, this is inappropriate for halfcap analysis. To determine the loads applied to the halfcap the load in each stringer is required to be obtained. This is achieved by restricting the movement of the vehicle in TIMBAR to one specific transverse position but installing false kerbs in the TIMBAR model.

By examination of the above diagram the largest bending loads in the halfcap are expected to be generated when one wheel pair is located directly over stringer 6 (which is 0.77 m from loading pile 4 centreline and therefore likely to load the halfcaps fully in shear). False kerbs in this example are therefore located to ensure that the wheel loads are directly over stringer 6.

As such the geometry for false kerbs to restrict vehicle movement is:

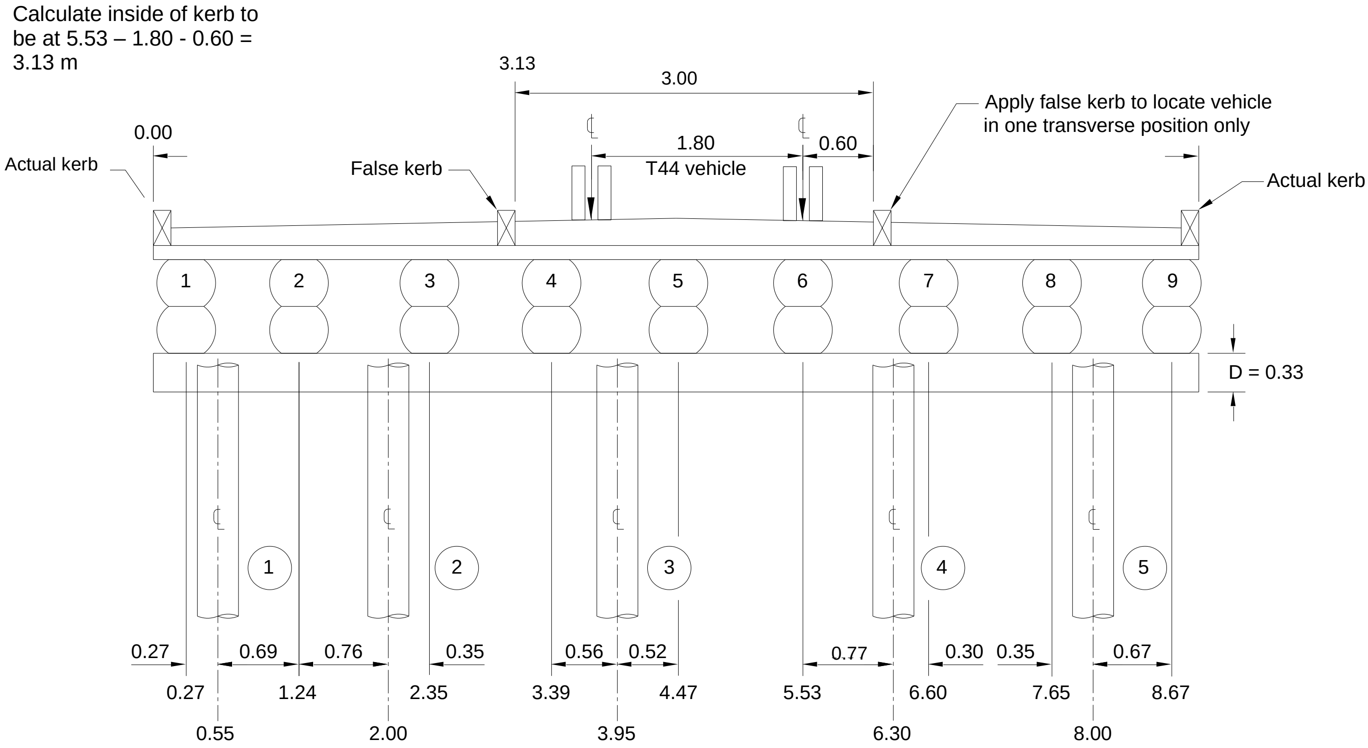


Figure C1 - Pier - Cross Section (Scale - NTS)

With the given geometry of the pier the proportion of the stringer loads taken by the halfcap can be determined as per methodology outlined at Section 1.4.2 “Halfcaps”.

Piles 1,3 & 5	0.450 m Diameter (Dia)	$\frac{Dia}{2}+\frac{D}{4}=\frac{0.450}{2}+\frac{0.330}{4}=0.308m$ $\frac{Dia}{2}+\frac{5D}{4}=\frac{0.450}{2}+\frac{5\times0.330}{4}=0.638m$
Pile 4	0.440 m Diameter (Dia)	$\frac{Dia}{2}+\frac{D}{4}=\frac{0.440}{2}+\frac{0.330}{4}=0.303m$ $\frac{Dia}{2}+\frac{5D}{4}=\frac{0.440}{2}+\frac{5\times0.330}{4}=0.633m$
Pile 2	0.410 m Diameter (Dia)	$\frac{Dia}{2}+\frac{D}{4}=\frac{0.410}{2}+\frac{0.330}{4}=0.288m$ $\frac{Dia}{2}+\frac{D}{4}=\frac{0.410}{2}+\frac{5\times0.330}{4}=0.618m$

Given the above D/4 and 5D/4 values the proportion of the stringer loads taken by the halfcap in shear and bending can now be determined.

Stringer No.	Closest Pile	Distance (Pile to Stringer) (m)	Stringer Within D/4?	Stringer Within 5D/4?		Proportion of load transferred to Halfcap in	
						Shear	Bending
1	1	0.270	Yes	Yes		0%	0%
2	1	0.690	No	No		100%	100%
3	2	0.350	No	Yes		18.8% *1	100%
4	3	0.560	No	Yes		76.4% *2	100%
5	3	0.520	No	Yes	→	64.2% *3	100%
6	4	0.770	No	No		100%	100%
7	4	0.300	Yes	Yes		0%	0%
8	5	0.350	No	Yes		12.7% *4	100%
9	5	0.670	No	No		100%	100%

Summary:

- Stringers 4 and 5 are within 5D/4 from the face of Pile 3, shear loads are therefore transferred to Pile 3 partially through direct bearing and partially through shear in halfcaps.
- Stringers 2, 6 and 9 are greater than 5D/4 from the face of the closest Pile, shear loads are therefore transferred to the Pile only by shear in halfcaps, with no direct bearing component onto the Pile.
- Bending and Shear Loads from stringers 1 and 7 are transferred to Piles through direct bearing onto Piles only as they are within D/4 from the face of the pile.
- Shear Loads from stringers 3, 4, 5 and 8 are transferred to Piles through a combination of shear and direct bearing.

*1 Stringer 3 is 0.350 m from centreline Pile 2

$$\therefore \text{ \% of load in shear } = \left(\frac{0.350 - 0.288}{0.618 - 0.288}\right) \times 100 = 18.8\%$$

*2 Stringer 4 is 0.560 m from centreline Pile 3

$$\therefore \text{ \% of load in shear } = \left(\frac{0.560 - 0.308}{0.638 - 0.308}\right) \times 100 = 76.4\%$$

*3 Stringer 5 is 0.520 m from centreline Pile 3

$$\therefore \text{ \% of load in shear } = \left(\frac{0.520 - 0.308}{0.638 - 0.308}\right) \times 100 = 64.2\%$$

*4 Stringer 8 is 0.350 m from centreline Pile 5

$$\therefore \text{ \% of load in shear } = \left(\frac{0.350 - 0.308}{0.638 - 0.308}\right) \times 100 = 12.7\%$$

The TIMBAR Model is now created for Spans 1 and 2 using the false kerb geometry shown previously. This allows the loads applied to the each halfcap, at the pier in question, from the stringers to be evaluated. To do this results from the span analysis from the TIMBAR analysis is used to determine maximum stringer loads applied to the halfcaps, which are then modified as per the load distribution above.

Since there is two halfcaps at pier 1 the most critical of these halfcaps is required to be analysed. As it is assumed that each of the halfcaps has the same properties, the only difference therefore is the applied loads. By visual inspection of the TIMBAR analysis results it can be seen that the loads applied to pier from Span 2 (Shear 1) are greater than those from Span 1 (Shear 2).

As we are analysing the pier 1 halfcap only ⅔ of shears quoted are used as it is assumed that all loads from one span are distributed ⅔ to the halfcap on that side and ⅓ to halfcap on the other side of the pier. No live load is assumed to be on the adjacent span.

Use Span 2 Shear 1 results from TIMBAR output (see pages C11 to C13).

Stringer	1	2	3	4	5	6	7	8	9
Dead Load (kN)	18.00	15.90	19.30	22.60	19.30	18.20	18.10	24.00	13.20
T44 (kN)	0.00	1.70	11.80	63.40	43.90	75.20	17.30	4.20	0.00
M1600 (kN)	0.00	3.10	21.40	97.20	57.00	107.60	29.40	8.30	0.00
Modified for ⅔:⅓ distribution and Dynamic Load Allowance									
⅔x1.3 T44	0.00	1.47	10.23	54.95	38.05	65.17	14.99	3.64	0.00
⅔x1.35 M1600	0.00	2.80	19.30	87.50	51.30	96.80	26.50	7.50	0.00

Shear Loads - Proportion of stringer loads producing shear in the halfcap

Stringer	1	2	3	4	5	6	7	8	9
Actual Loads (inc. DLA)									
Dead (kN)	18.0	15.90	19.30	22.60	19.30	18.20	18.10	24.00	13.20
T44 (kN)	0.00	1.47	10.23	54.95	38.05	65.17	14.99	3.64	0.00
M1600 (kN)	0.00	2.80	19.30	87.50	51.30	96.80	26.50	7.50	0.00
Percentage of load transferred to Halfcap	0%	100%	18.8%	76.4%	64.2%	100%	0%	12.7%	100%
Revised Loads (inc. DLA)									
Dead Load (kN)	0.00	15.90	3.63	17.27	12.39	18.20	0.00	3.05	13.20
T44 (kN)	0.00	1.47	1.92	41.98	24.43	65.17	0.00	0.46	0.00
M1600 (kN)	0.00	2.80	3.63	66.85	32.93	96.80	0.00	0.95	0.00

Reduced loads graphically represented (decimal places ignored for clarity) as:

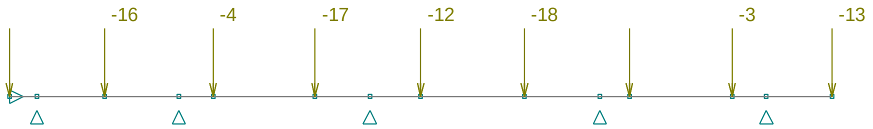


Figure C2 - Dead Loads

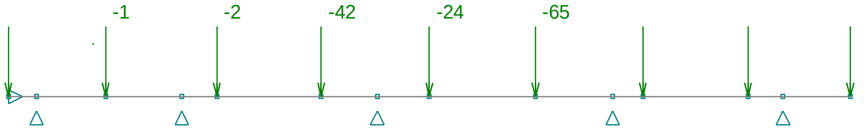


Figure C3 - T44 Loads

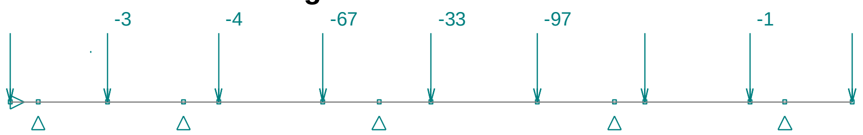


Figure C4 - M1600 Loads

Bending Loads - Proportion of stringer loads producing bending in the halfcap

Stringer	1	2	3	4	5	6	7	8	9
Actual Loads (inc. DLA)									
Dead (kN)	18.00	15.90	19.30	22.60	19.30	18.20	18.10	24.00	13.20
T44 (kN)	0.00	1.47	10.23	54.95	38.05	65.17	14.99	3.64	0.00
M1600 (kN)	0.00	2.80	19.30	87.50	51.30	96.80	26.50	7.50	0.00
Percentage of load transferred to Halfcap	0%	100%	100%	100%	100%	100%	0%	100%	100%
Revised Loads (inc. DLA)									
Dead Load (kN)	0.00	15.9	19.3	22.6	19.3	18.2	0.00	24.00	13.20
T44 (kN)	0.00	1.47	10.23	54.95	38.05	65.17	0.00	3.64	0.00
M1600 (kN)	0.00	2.8	19.3	87.5	51.3	96.8	0.00	7.50	0.00

Reduced loads graphically represented (decimal places ignored for clarity) as:

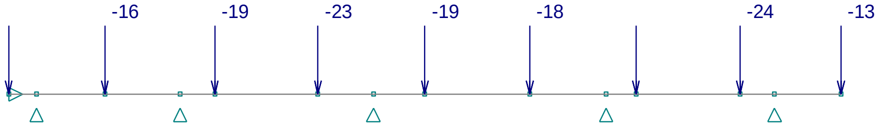


Figure C5 - Dead Loads

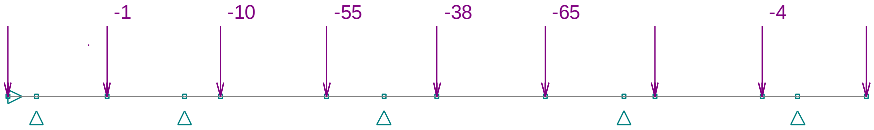


Figure C6 - T44 Loads

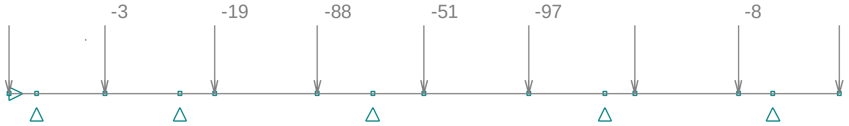


Figure C7 - M1600 Loads

Calculation of Forces (Bending/Shear) in Halfcap

This is the Bending Moments and Shear Forces that are generated in the halfcap as a result of the above reduced Bending and Shear loads from the stringers.

These loads are derived by modelling the halfcap as a continuous or discontinuous member, as appropriate (see Section 1.4.3 'Deck Planks' for guidelines), in an appropriate software package to determine actual loads in halfcap.

Shear Results (decimal places ignored for clarity):

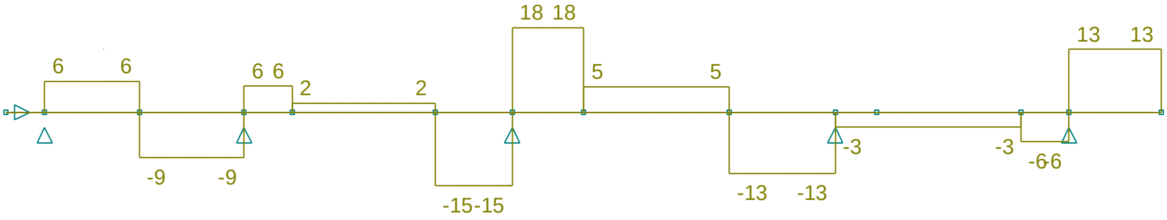


Figure C8 - Dead Loads

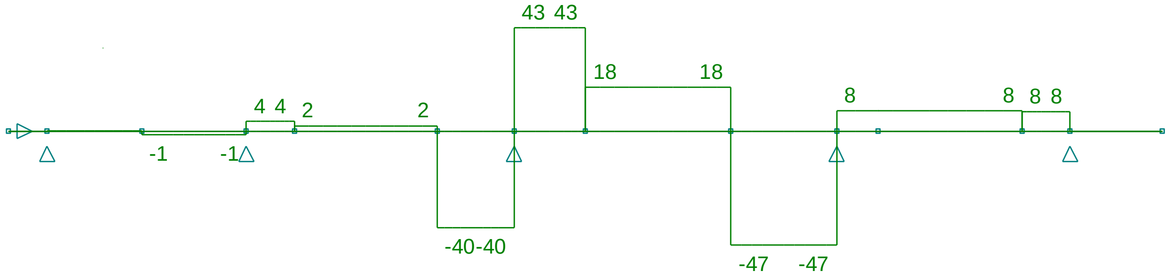


Figure C9 - T44 Vehicle Loads

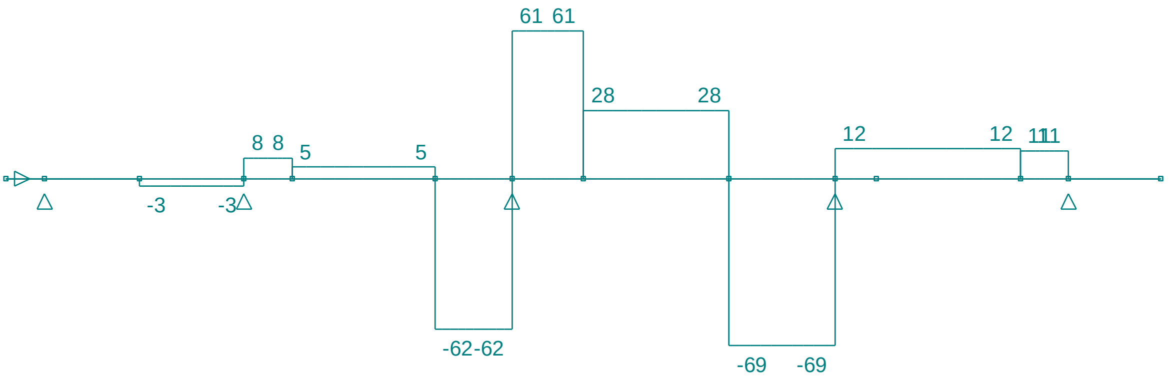


Figure C10 - M1600 Vehicle Loads

Shear Force at Pile 4 is critical

- Dead Load = 13 kN
- T44 Load = 47 kN
- M1600 Load = 69 kN

$$f_{s..(DL)} = \frac{13000}{170 \times 330} = 0.23\text{MPa} \quad \text{(average shear stress)}$$

$$f_{s..(1.3T44)} = \frac{47000}{170 \times 330} = 0.84\text{MPa} \quad \text{(average shear stress)}$$

$$f_{s..(1.35M1600)} = \frac{69000}{170 \times 330} = 1.23\text{MPa} \quad \text{(average shear stress)}$$

Shear Capacity:

$$F_s = k_1 \frac{2F'_S}{3} = 1.40 \times \frac{2 \times 1.25}{3} = 1.17\text{MPa} \quad \text{(Main Road – } k_1 = 1.40\text{)}$$

(F_S is max allowable stress and $\frac{2}{3}F'_S$ is average allowable stress)

(F'_S = 1.25 MPa for F14 Jarrah Halfcaps)

Rating:

$$\%T44 = \left(\frac{1.17 - 0.23}{0.84}\right) \times 100 = 112\% \rightarrow \text{Adequate Capacity}$$

$$\%M1600 = \left(\frac{1.17 - 0.23}{1.23}\right) \times 100 = 76.4\% \rightarrow \text{Insufficient Capacity}$$

Bending Results (decimal places ignored for clarity):

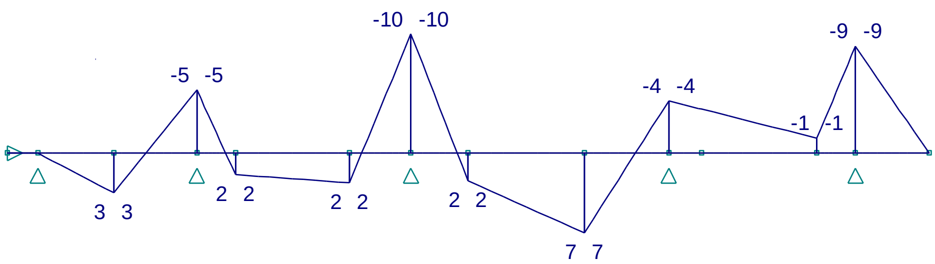


Figure C11 - Dead Loads

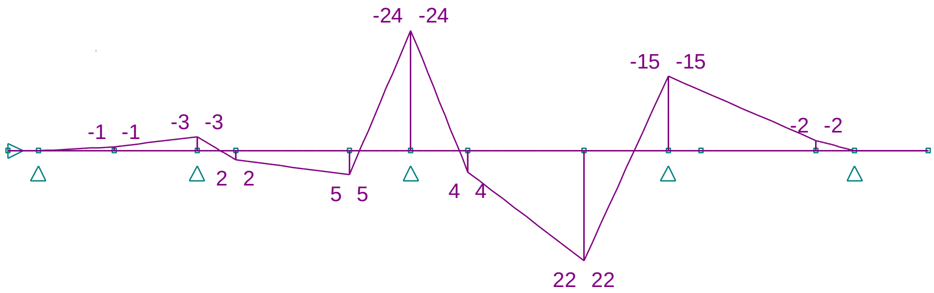


Figure C12 - T44 Vehicle Loads

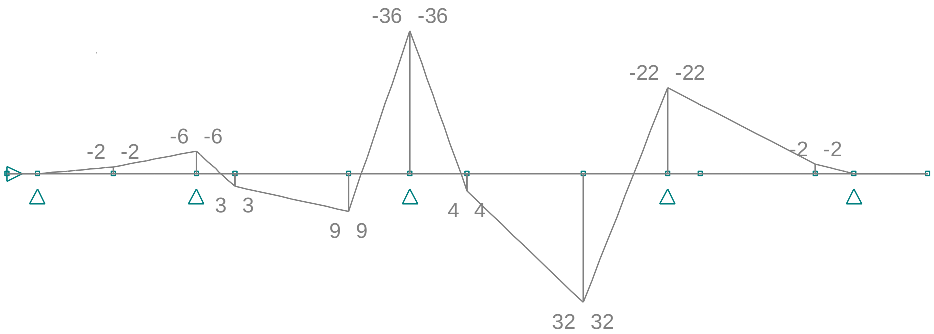


Figure C13 - M1600 Vehicle Loads

Bending Moment at Pile 3 is critical

- Dead Load = 10 kNm
- T44 Load = 24 kNm
- M1600 Load = 36 kNm

$$f_{b..(DL)} = \frac{M}{Z} = \frac{6M}{bd^2} = \frac{6 \times 10 \times 10^6}{170 \times 330^2} = 3.24\text{MPa (average bending stress)}$$

$$f_{b..(1.3T44)} = \frac{6 \times 24 \times 10^6}{170 \times 330^2} = 7.78\text{MPa (average bending stress)}$$

$$f_{b.(1.35M1600)} = \frac{6 \times 36 \times 10^6}{170 \times 330^2} = 11.67\text{MPa (average bending stress)}$$

Bending Capacity:

$$F_b = k_1 F'_b = 1.40 \times 14.0 = 19.6\text{MPa} \text{ (Main Road} - k_1 = 1.40)$$

(F_b = 14.0 MPa for F14 Jarrah Halfcaps)

Rating:

$$\%T44 = \left(\frac{19.6 - 3.24}{7.78}\right) \times 100 = 210\% \rightarrow \text{Adequate Capacity}$$

$$\%M1600 = \left(\frac{19.6 - 3.24}{11.67}\right) \times 100 = 140\% \rightarrow \text{Adequate Capacity}$$

In summary, as a result of the above assessment it is evident that the existing Jarrah (F14) halfcap has sufficient capacity in both bending and shear to withstand T44 vehicle loads. It has though been found that the halfcap shear capacity is only sufficient enough to withstand 76% of a M1600 design vehicle.

Note:

1. Such a rating (less than 100%) of the M1600 vehicle does not typically represent a deficiency in the bridge as the M1600 vehicle does is a theoretical vehicle and does not represent a real road vehicle. The rating of this vehicle is used as a guide only and should not be used to justify the scope of works that is required to be undertaken.
2. A similar approach to that taken above for the rating of T44 and M1600 vehicles can be undertaken for any other vehicle configuration that has been analysed using TIMBAR.

TIMBAR Output (3 pages)

With false kerbs installed as per page C3

Description		Bridge No. 324 Span 2 (324DB)								
Stringer Number	Unit	1	2	3	4	5	6	7	8	9
INSPECTION DATA										
Stringer Material		Jarrah	Jarrah	Jarrah	Jarrah	Jarrah	Jarrah	Jarrah	Jarrah	Jarrah
Steel Stringer Type										
Stringer Grade		Default	Default	Default	Default	Default	Default	Default	Default	Default
End 1										
Net Area End 1	mm^2	1.38E+05	1.58E+05	1.49E+05	1.79E+05	1.47E+05	1.49E+05	1.52E+05	2.03E+05	1.24E+05
Iyy End 1	mm^4	1.24E+09	1.41E+09	1.34E+09	1.84E+09	1.32E+09	1.34E+09	1.36E+09	2.75E+09	1.12E+09
Midspan										
Gross Area Midspan	mm^2	2.28E+05	1.74E+05	2.03E+05	2.21E+05	1.90E+05	1.75E+05	2.05E+05	2.55E+05	1.79E+05
Iyy Midspan	mm^4	3.55E+09	2.10E+09	2.80E+09	3.03E+09	2.74E+09	2.66E+09	3.21E+09	4.63E+09	2.48E+09
Y Max Midspan	mm	250.60	221.30	235.70	234.90	241.90	248.40	251.70	270.70	237.20
End 2										
Net Area End 2	mm^2	1.70E+05	1.27E+05	1.78E+05	2.13E+05	1.87E+05	1.32E+05	1.87E+05	1.70E+05	1.55E+05
Iyy End 2	mm^4	1.59E+09	1.15E+09	1.58E+09	3.31E+09	2.11E+09	1.20E+09	2.11E+09	1.59E+09	1.38E+09

MATERIAL PROPERTIES										
Road Type		MR	MR	MR	MR	MR	MR	MR	MR	MR
k1		1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40
Bending Capacity										
F'b	MPa	17	17	17	17	17	17	17	17	17
Midspan Condition (G = "good", F = "friable", R = "rot")		G	G	G	G	G	G	G	G	G
Fb	MPa	23.8	23.8	23.8	23.8	23.8	23.8	23.8	23.8	23.8
Shear Capacity										
F's	MPa	1.45	1.45	1.45	1.45	1.45	1.45	1.45	1.45	1.45
Average Stress Factor		0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66
End 1 Condition		G	G	G	G	G	G	G	G	G
End 1 Fs	MPa	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34
End 2 Condition		G	G	G	G	G	G	G	G	G
End 2 Fs	MPa	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34	1.34

Description		Bridge No. 324 Span 2 (324DB)								
Stringer Number	Unit	1	2	3	4	5	6	7	8	9
DEAD LOADS										
Dead Load Moment	kNm	27.15	20.58	28.06	33.10	27.83	24.50	26.97	34.10	18.81
Dead Load Shear End 1	kN	17.96	15.92	19.27	22.58	19.33	18.20	18.09	24.04	13.16
Dead Load Shear End 2	kN	18.57	15.14	19.41	23.67	20.19	16.70	20.38	21.96	14.01
LIVE LOADS										
T44 Moment	kNm	0.00	2.36	19.92	56.47	58.19	56.75	29.70	5.78	0.00
T44 Shear 1	kN	0.00	1.66	11.84	63.42	43.92	75.17	17.26	4.23	0.00
T44 Shear 2	kN	0.00	1.82	10.79	59.92	44.06	67.46	18.94	3.50	0.00
M Truck Moment	kNm	0.00	1.02	9.33	30.94	29.60	32.66	13.97	2.56	0.00
M Truck Shear 1	kN	0.00	0.71	4.76	30.71	15.10	39.23	6.76	1.76	0.00
M Truck Shear 2	kN	0.00	0.76	4.46	29.10	15.66	36.34	7.43	1.43	0.00
Tandem Moment	kNm	0.00	2.21	18.68	52.95	54.55	53.20	27.84	5.42	0.00
Tandem Shear 1	kN	0.00	1.56	11.10	58.92	40.59	70.05	16.18	3.96	0.00
Tandem Shear 2	kN	0.00	1.71	10.11	55.74	40.81	62.93	17.76	3.28	0.00
Triaxle Moment	kNm	0.00	3.17	26.50	70.57	75.58	69.19	39.89	7.92	0.00
Triaxle Shear 1	kN	0.00	1.83	14.48	66.28	49.92	77.98	20.73	4.82	0.00
Triaxle Shear 2	kN	0.00	2.03	13.38	63.29	50.53	69.75	22.83	3.84	0.00
Quadaxle Moment	kNm	0.00	3.53	28.87	75.17	81.28	72.89	43.40	8.79	0.00
Quadaxle Shear 1	kN	0.00	2.06	16.72	73.14	58.05	83.74	24.20	5.48	0.00
Quadaxle Shear 2	kN	0.00	2.31	15.24	70.15	58.79	74.83	26.67	4.36	0.00
484-Quad Moment	kNm	0.00	2.66	20.88	57.30	60.13	56.65	31.32	6.63	0.00
484-Quad Shear 1	kN	0.00	1.48	12.56	63.98	46.55	74.30	18.27	3.94	0.00
484-Quad Shear 2	kN	0.00	1.69	11.27	60.73	46.82	66.65	20.12	3.13	0.00
M1600 Moment	kNm	0.00	5.51	39.42	99.13	99.59	93.11	57.75	14.14	0.00
M1600 Shear 1	kN	0.00	3.11	21.40	97.22	56.99	107.58	29.42	8.33	0.00
M1600 Shear 2	kN	0.00	3.37	19.85	92.99	59.40	96.44	32.42	6.80	0.00
SECTION CAPACITIES										
Bending Moment Capacity										
I midspan	mm^4	3.55E+09	2.10E+09	2.80E+09	3.03E+09	2.74E+09	2.66E+09	3.21E+09	4.63E+09	2.48E+09
y max	mm	251	221	236	235	242	248	252	271	237
Maximum Moment	kNm	337	226	282	307	269	255	304	407	248
Dead Load Moment	kNm	27	21	28	33	28	24	27	34	19
Max. Live Load Moment	**kNm**	**310**	**205**	**254**	**273**	**242**	**230**	**277**	**373**	**230**
Shear Capacity End 1										
Net Area End 1	mm^2	1.38E+05	1.58E+05	1.49E+05	1.79E+05	1.47E+05	1.49E+05	1.52E+05	2.03E+05	1.24E+05
Maximum Shear Force	kN	185	212	200	240	196	200	204	271	166
Dead Load Shear Force	kN	18	16	19	23	19	18	18	24	13
Max. Live Load Shear	**kN**	**167**	**196**	**181**	**217**	**177**	**182**	**186**	**247**	**153**
Shear Capacity End 2										
Net Area End 2	mm^2	1.70E+05	1.27E+05	1.78E+05	2.13E+05	1.87E+05	1.32E+05	1.87E+05	1.70E+05	1.55E+05
Maximum Shear Force	kN	227	170	238	285	251	177	251	227	208
Dead Load Shear Force	kN	19	15	19	24	20	17	20	22	14
Max. Live Load Shear	**kN**	**209**	**154**	**219**	**262**	**231**	**161**	**230**	**205**	**194**

Description		Bridge No. 324 Span 2 (324DB)								
Stringer Number	Unit	1	2	3	4	5	6	7	8	9
LOAD RATINGS										
T44	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	10488320.10	2946.71	432.40	164.05	140.68	137.46	315.89	2185.22	7778272.15
Shear End 1	Tonne	5654596.47	3989.76	517.03	115.90	136.49	81.94	364.66	1980.66	5168189.47
Shear End 2	Tonne	7063591.25	2868.84	687.48	147.88	177.31	80.62	412.06	1985.93	6559977.42
Limiting	**Tonne**	**5654596.47**	**2868.84**	**432.40**	**115.90**	**136.49**	**80.62**	**315.89**	**1980.66**	**5168189.47**
%T44 Load	**%**	**12840646%**	**6515%**	**982%**	**263%**	**310%**	**183%**	**717%**	**4498%**	**11736097%**
M Truck	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	2381722.69	1541.07	209.71	67.99	62.81	54.23	152.49	1121.94	1766315.97
Shear End 1	Tonne	1284064.62	2116.57	292.42	54.35	90.14	35.65	211.46	1078.90	1173609.69
Shear End 2	Tonne	1604023.85	1565.58	377.53	69.15	113.27	33.99	238.50	1107.33	1489661.54
Limiting	**Tonne**	**1284064.62**	**1541.07**	**209.71**	**54.35**	**62.81**	**33.99**	**152.49**	**1078.90**	**1173609.69**
Tandem	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	4287100.84	1284.62	188.53	71.52	61.34	59.93	137.73	952.72	3179368.74
Shear End 1	Tonne	2311316.31	1739.75	225.42	50.99	60.37	35.94	158.99	863.72	2112497.45
Shear End 2	Tonne	2887242.92	1250.82	299.76	64.98	78.25	35.33	179.67	865.68	2681390.77
Limiting	**Tonne**	**2311316.31**	**1250.82**	**188.53**	**50.99**	**60.37**	**35.33**	**137.73**	**863.72**	**2112497.45**
Triaxle	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	6430651.26	1343.62	199.31	80.49	66.41	69.12	144.20	978.12	4769053.11
Shear End 1	Tonne	3466974.46	2226.40	259.22	67.99	73.62	48.43	186.17	1065.27	3168746.17
Shear End 2	Tonne	4330864.38	1578.91	339.83	85.83	94.80	47.81	209.59	1109.21	4022086.15
Limiting	**Tonne**	**3466974.46**	**1343.62**	**199.31**	**67.99**	**66.41**	**47.81**	**144.20**	**978.12**	**3168746.17**
Quadaxle	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	8574201.68	1606.86	243.90	100.76	82.33	87.49	176.72	1174.48	6358737.49
Shear End 1	Tonne	4622632.62	2636.29	299.39	82.15	84.41	60.13	212.65	1250.01	4224994.89
Shear End 2	Tonne	5774485.85	1853.46	397.94	103.26	108.62	59.42	239.26	1303.19	5362781.54
Limiting	**Tonne**	**4622632.62**	**1606.86**	**243.90**	**82.15**	**82.33**	**59.42**	**176.72**	**1174.48**	**4224994.89**
484-Quad	DLA	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Bending	Tonne	8574201.68	2133.63	337.28	132.19	111.30	112.56	244.90	1556.41	6358737.49
Shear End 1	Tonne	4622632.62	3655.58	398.54	93.92	105.26	67.77	281.71	1736.63	4224994.89
Shear End 2	Tonne	5774485.85	2535.74	537.88	119.28	136.41	66.71	317.09	1814.31	5362781.54
Limiting	**Tonne**	**4622632.62**	**2133.63**	**337.28**	**93.92**	**105.26**	**66.71**	**244.90**	**1556.41**	**4224994.89**
M1600	DLA	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S	1.35B:1.35S
Bending	Tonne	30862740.60	3969.05	687.97	294.28	258.82	263.79	511.54	2811.32	22888202.63
Shear End 1	Tonne	16803584.75	6712.08	901.20	238.07	331.22	180.28	673.71	3166.43	15358144.52
Shear End 2	Tonne	20990649.79	4896.11	1176.69	300.05	414.15	177.58	758.20	3217.42	19494076.56
Limiting	**Tonne**	**16803584.75**	**3969.05**	**687.97**	**238.07**	**258.82**	**177.58**	**511.54**	**2811.32**	**15358144.52**
%M1600 Load	**%**	**11669156%**	**2756%**	**478%**	**165%**	**180%**	**123%**	**355%**	**1952%**	**10665378%**

The resulting maximum loads for each axle group for this span are:

Vehicle	**Load Rating (%)**	**Load Rating (Tonne)**	Typical Weight (Tonne)	Critical Stringer (No)	Critical Stress Type	Critical Section
T44	**183%**	**80.6**	**44**	6	Shear	End 2
M Truck	**340%**	**34.0**	**10**	6	Shear	End 2
Tandem	**196%**	**35.3**	**18**	6	Shear	End 2
Triaxle	**177%**	**47.8**	**27**	6	Shear	End 2
Quadaxle	**165%**	**59.4**	**36**	6	Shear	End 2
484-Quad	**185%**	**66.7**	**36**	6	Shear	End 2
M1600	**123%**	**177.6**	**144**	6	Shear	End 2

Bridge No. 324 Span 2 (324DB)

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APPENDIX D

Timber Bridge Strengthening and Refurbishment Design Process

Timber Bridge Strengthening and Refurbishment Design Process

Step 1: Data Gathering and Recording
Retrieve all relevant information and documents for the bridge, assess required geometry, load rate the bridge to its proposed geometry (and strengthening where applicable), prepare and complete all control documents and progressively sign off once milestones have been achieved.

Step 2: Preliminary Design
Based on Inspection Reports and load rating, prepare preliminary design scope and sketches, compare to previously budgeted scope, liaise with the Region regarding any differences between budgeted and proposed scopes and agree on any changes proposed.

Step 3: Site Visit and Liaison
Visit the bridge site, measure up and confirm bridge geometry and defective elements, agree proposed bridge strengthening and proposed railing extents while on site with Region and SDS.

Step 4: Preliminary Design Review
Prepare a 15% design report and sketches based on agreed scope of work and required strengthening, hold 15% review meeting, modify report and sketches as required to suit any agreed changes with SES and Region resulting from meeting and resubmit for signing off.

Step 5: Detailed Design
Carry out detailed design calculations and sketches as required for any non-standard details. Complete any additional structural analysis required for the refurbishment design and prepare instructions for drafting, including standard details proposed to be used.

Step 6: Drafting and Specifications
Drafting to commence once all design information, scope and sketches prepared. Designer to liaise closely with the Draftsperson during this phase. Once drawings have been checked by the designer and changes made, arrange for 85% Drafting Review and 100% Design Review meetings. Prepare specifications.