DESIGN AND ASSESSMENT OF NSW TIMBER BRIDGES

Guide

AUGUST 2021

About this release

Document information

Title:	Design and Assessment of NSW Timber Bridges
Reference No:	R11-B005-001
Version and date:	DRAFT (August 2021)
Prepared by:	Amie Nicholas, Heritage and Conservation Engineer (NPER)

Approval

Recommended by:		*Signature*	DD/MM/YY
Approved by:		*Signature*	DD/MM/YY

Revision history

Version	Date	Revision description
1.0		First issue

Important Disclaimer

This guide was prepared by a study team managed by officers of the Roads & Traffic Authority of New South Wales (RTA) and is for guidance purposes only. This guide was prepared solely for the RTA's internal purposes and does not take into account the specific circumstances or purposes of any other person. The opinions expressed in this guide are those of the study team and not necessarily those of the RTA. No person should rely on the contents of this guide without first obtaining advice from a qualified professional person in accordance with their specific circumstances. Those persons applying any contents, opinions or conclusions contained in this guide do so at their own risk. The RTA expressly disclaims all and any liability and responsibility to any person in respect of anything, and of the consequences of anything done, or omitted to be done by any person in reliance, whether wholly or partly, upon the contents of this guide.

Produced by the Roads and Traffic Authority of New South Wales.

Table of Contents

1 Introduction

1.1 Purpose of the guide

Timber bridges in New South Wales were once the indispensable technology of inland expansion. Timber truss bridges in particular are important functional elements of the built environments and scenery of local communities. As a collection, they created New South Wales's reputation as "the timber bridge state". They form a tangible record of a segment of the state's engineering history and the evolution of its road and transport system.

It is in the nature of infrastructure to become obsolete, and many of these bridges are now 'pinch-points' and 'bottle-necks', restraining a burgeoning national road transport system. Roads and Maritime Services (RMS) have a core responsibility to provide roads and bridges that safely carry traffic. It also has obligations under the Heritage Act and the Government's Total Asset Management policy for the stewardship of these assets and protection of their significance for future generations. This guide is to assist structural engineers in the rehabilitation design and assessment of these historically significant structures in order to meet RMS obligations.

1.2 Application to RMS projects

The principal focus of this guide is the rehabilitation design and structural assessment of existing timber bridges, or bridges with significant timber components. This guide applies especially to bridges of heritage significance, which are listed on RMS' Section 170 Register.

1.3 Who should use this guide?

Structural Engineers responsible for the rehabilitation design or structural assessment of heritage timber bridges should use this guide. The principles in the guide may also be useful for external organisations including councils, consultants and contractors.

1.4 Relationship to other RMS documents

The guide integrates with other RMS documents and systems. The process of design development for heritage bridges is iterative by nature, involving a large number of stakeholders as outlined in the Rehabilitation Design QA System (Technical Procedure OTB-TP-301).

Decisions regarding the retention or otherwise of RMS timber truss bridges are covered in the RMS' Timber Truss Bridge Conservation Strategy which was endorsed by the Heritage Council on 4 July 2012. This strategy outlines RMS' assessment of the future operational requirements and limitations of these bridges, and defines the 26 bridges to be retained (some to be conserved with original design details, and many to be strengthened in ways sympathetic to their heritage significance) to represent more than 420 timber truss bridges built in NSW in the 1850s to 1930s.

2 Significance of Timber Bridges

2.1 Australia's Unique Hardwood Timbers

When Europeans first explored Australia, they were less than impressed by the Australian timbers. Captain James Cook said in 1770 that the trees were so "hard and ponderous" that they were pretty much useless. Surgeon John White reported in 1790 that, "I do not know any one purpose for which it (Australian timber) will answer except for firewood; and for that it is excellent; but in other respects it is the worst wood that any country or climate ever produced."

Various newspaper articles of the late 1700s and very early 1800s describe the difficulties the convicts had in dealing with the Australian timbers due to their "monstrous bulk", hardness and incredible weight. The trees in the immediate vicinity of the settlement at Sydney were too crooked, too hard to work, and too damaged by fire to be used as a structural material.

However, it wasn't long before timbers were discovered in Australia which would rival any in the world. Red Cedar was discovered in the Hawkesbury Flats and gangs of convicts were immediately sent to cut them down. Sixty logs from the Hawkesbury were exported to India as early as 1795, followed by loads to England, China, South Africa and New Zealand. In Australia the timber was used to build houses, barns, rough bush furniture as well as very fine furniture. It built pigsties, cow sheds, paling fences, railway sleepers and, of course, bridges.

Between 1855 and 1886, there were international exhibitions of timber in Paris, Melbourne, London, Sydney and New Zealand. The judges sawed the samples, planed them, nailed them and tested them for strength. Australian timbers met high praise. Experiments were made at the foundry of P.N. Russell & Co. in 1860 which showed how much tougher the ironbark is than Baltic or American timber. The conclusion made was that whatever span had been possible with timber in other countries could certainly be imitated, if not surpassed, in New South Wales.

Figure 1: Forest of young Black-Butt Trees (1896)

Figure 2: Tallow-wood Logs for Transport (1896)

In 1896, J. J. C. Bradfield, famous for the design of the Sydney Harbour Bridge, reported on the comparative strength of ironbark and iron, and found that, for the same weight, ironbark is more than three times stronger than iron in tension, and almost twice as strong as iron in compression.

In 1871 members of parliament thought it time to begin saving the trees, and the first reserves were designated, 'to protect some of the magnificent forests of brush and hardwood in the Clarence Pastoral Districts, and the flooded red gum forests on the Murray River'. Unfortunately, the Clarence declaration did not save the Big Scrub on the north bank of the Richmond River, which had contained at least 50,000 hectares of some of the finest timber in the world, but clearing began seriously in the 1880's, and by 1900, the forest had disappeared.

Around this time, the duty of inspecting exported timber fell to the Department of Public Works. It was thought that, whatever views may be held as to the advisableness of sending away large quantities of our best timbers, it was desirable that all such exports should be properly inspected and classed. By 1904, the rapid disappearance of hardwoods was increasing due to the recognition of its value by the commercial world of Europe, South Africa, and the East. In 1907 it was reported that excessive exports had greatly increased the price of timber, and that unless there be some check given to the trade, national works were likely to be seriously handicapped. Practically nothing was thought to have been done towards replenishing the enormous drain represented by the constant demand for railway sleepers, bridge, wharf, and building timbers.

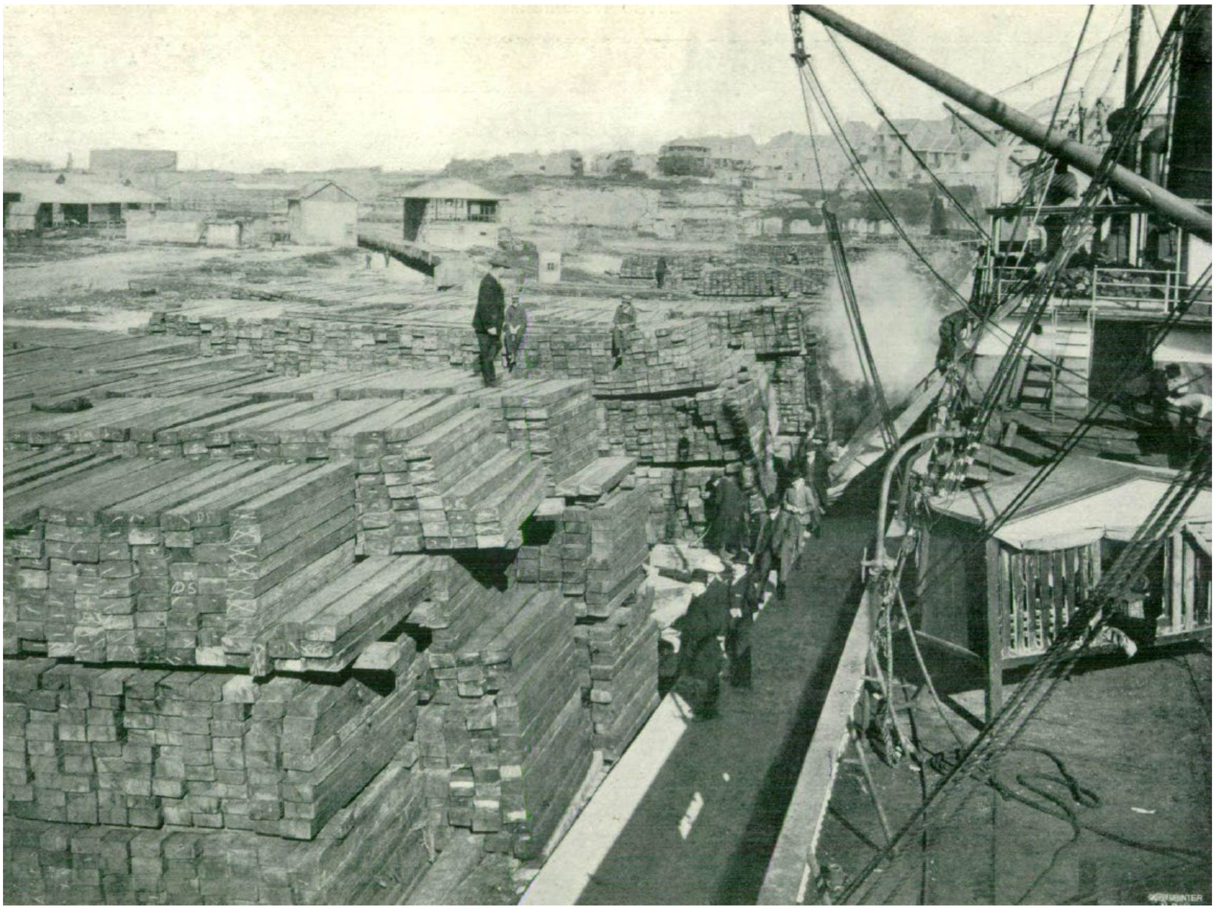


Figure 3: Sleepers and Girders being loaded at Darling Harbour for South Africa in 1903

Percy Allan rather discourteously described the difficulties in obtaining large long lengths of timber for McDonald trusses in 1895: "Again, some of the flitches are 53' 6" long and, having to be free of heart and sapwood, are difficult to obtain, and this oftentimes occasioned delay in the erection of the structures, the simple-minded sawmill proprietor supplying all the short and profitable sizes in the bridge, and then pleading inability to supply the more costly flitches."

It would seem that saw millers had something of a reputation, as seen by Henry Kendall's poem below. Thomas Henry Kendall (1839-1882) was born in Ulladulla, New South Wales, and was once regarded as Australia's finest poet, and is known for his distinctly Australian poetry. Not only was Kendall a poet, but he also worked for a time in the timber business in the Mid North Coast of NSW, and was, for the last 18 months of his life, appointed by Henry Parkes as inspector of forests, for which he was admirably fitted by his knowledge of native timbers.

JIM THE SPLITTER, by Henry Kendall

No party is Jim of the Pericles type —
He is modern right up from the toe to the pipe;
And being no reader or roamer,
He hasn't Euripides much in the head;
And let it be carefully, tenderly said,
He never has analysed Homer…

You mustn't, however, adjudge him in haste,
Because a red robber is more to his taste
Than Ruskin, Rossetti, or Dante!
You see, he was bred in a bangalow wood,
And bangalow pith was the principal food
His mother served out in her shanty.

His knowledge is this — he can tell in the dark
What timber will split by the feel of the bark;
And rough as his manner of speech is,
His wits to the fore he can readily bring
In passing off ash as the genuine thing
When scarce in the forest the beech is.

In girthing a tree that he sells "in the round,"
He assumes, as a rule, that the body is sound,
And measures, forgetting to bark it!
He may be a ninny, but still the old dog
Can plug to perfection the pipe of a log
And "palm it" away on the market.

He splits a fair shingle, but holds to the rule
Of his father's, and, haply, his grandfather's school;
Which means that he never has blundered,
When tying his shingles, by slinging in more
Than the recognized number of ninety and four
To the bundle he sells for a hundred!

When asked by the market for ironbark red,
It always occurs to the Wollombi head
To do a "mahogany" swindle.
In forests where never the ironbark grew,
When Jim is at work, it would flabbergast you
To see how the "ironbarks" dwindle…

He shines at his best at the tiller of saw,
On the top of the pit, where his whisper is law
To the gentleman working below him.
When the pair of them pause in a circle of dust,
Like a monarch he poses — exalted, august —
There's nothing this planet can show him!

… So much for our hero! A statuesque foot
Would suffer by wearing that heavy-nailed boot —
Its owner is hardly Achilles.
However, he's happy! He cuts a great "fig"
In the land where a coat is no part of the "rig" —
In the country of damper and "billies."

2.2 Early Australian Timber Bridges

The first bridge to be built in Australia was in 1788 when a gang of convicts were employed in rolling timber together to form a bridge over the Tank Stream in Sydney. This bridge lasted more than 15 years until it was replaced in 1804 by a "more permanent" stone arch bridge, which collapsed within twelve months and had to be rebuilt. The stone bridge was again largely rebuilt in 1811 at a cost of '660 gallons of spirits'. The idea that timber bridges are "temporary" structures has been pervasive throughout their history, despite many of them outlasting so called "more permanent" structures made of "modern" materials such as steel and concrete.

This is clearly indicated in the report to the Legislative Assembly of New South Wales of the Department of Public Works in 1897, which states, "With regard to the repairs and maintenance of bridges, which now demand a large and yearly-increasing expenditure, the Assistant Engineer suggests, as settlement advances in the Colony, replacing timber structures, so far as practicable, by bridges of a more permanent character, and thus reducing the annual cost of repairs and maintenance. He points out that, in consequence of the improvement effected of late years to the surface of the roads, and the cutting down of grades, the bridges are now required to bear the strain of much heavier loads than they were estimated to sustain at the time they were built."

Percy Allan, the first Australian born engineer to be appointed Chief Bridge Engineer, was also affectionately known as "Mr Timber Truss Bridge", and was synonymous with timber bridges. He challenged the popular idea that steel bridges were more economical in the long run than timber, arguing in 1924 that this idea was based on overseas experience with lesser quality timber. He said that, "In Australia, however, with timber bridges of modern design built of more durable hardwood, experience has shown that the popular idea has no solid foundation in fact."

Despite this, it is true that the early days of timber bridge building in Australia were largely experimental, and not always terribly successful. Until the 1850s when gold was discovered in NSW, settlement was largely confined to a narrow coastal strip, and transport was generally by river rather than road. Carpenters found that their axes were incapable of handling the NSW hardwoods and it took time to develop tools and methods for Australian timbers.

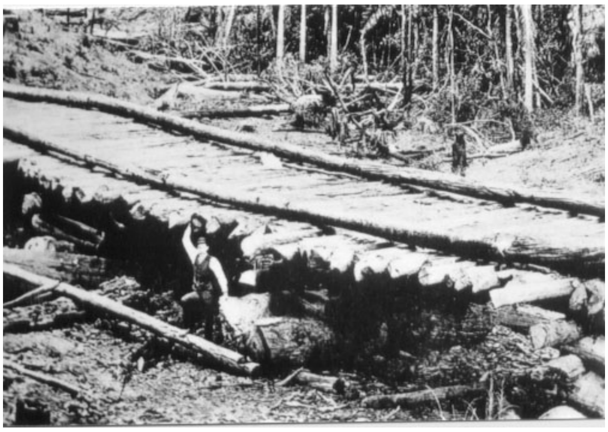


Figure 4: Early Bridges: A Corduroy Bridge

Figure 5: Early Bridges: A Turf Covered Bridge

At the beginning of the second half of the 19th century in colonial New South Wales, the need for better road transport had become urgent. Trade and commerce was being stifled and goods damaged at the prevailing river fords. Travel generally too was slow, uncomfortable and potentially dangerous, and the movement of people had dramatically increased with the Gold Rushes of the early 1850s. The New South Wales Government saw a need for a better road system because a significant amount of its rural wealth was being exported via the inland river system through the rival ports of Melbourne and from Goolwa in South Australia.

The first timber arch bridge was built in Maitland in 1852. Timber arches were popular at first for both road and rail bridges but fabrication was difficult, and they were subject to deterioration and distortion, and so this type of bridge did not last very long, and none remain today. The main problem was the separation of the laminates, due to the large amount of shrinkage of the Australian hardwoods, and the consequent penetration of water into the joints. Once fungi or termites attacked the timber it was impossible to renew the laminates or portions of the arch. These bridges were costly to build, and as their short lives proved, they were not cost-effective.

Figure 6: A Laminated Timber Arch Bridge over South Creek at Windsor

Despite the early difficulties with timber as a structural material in NSW, engineers continued to experiment, and the first timber truss bridge in NSW was built in Carcoar between Bathurst and Cowra in 1856. Although good work was done by the early colonial road-engineers, the real engineering history of NSW dates from the formation of the Public Works Department (PWD) in 1858 shortly after the inauguration of responsible Government in the Colony. NSW could not afford to build costly masonry arches or use expensive imported iron for its bridges. Despite the gold rushes of the 1850s, the returns to NSW were less than a quarter of Victoria's and yet its land area was more than four times that of Victoria. In 1861, just after the timber experiments at the foundry of P.N. Russell & Co., there was a government decree that local materials and skills were to be used as much as possible in order to minimise expensive imports of iron.

2.3 Construction of Timber Bridges

Following are excerpts from Lynn Heather Mackay's thesis (Timber Truss Bridges in New South Wales, Bachelor Architecture, University of Sydney 1972) regarding timber bridge construction:

> "Construction of timber truss bridges was generally carried out by families of bridge carpenters who travelled from site to site with a bullock or horse team, sometimes with as many as fifty horses. In the nineteenth and early twentieth centuries, all moving was done in this way, and there were very few mechanical aids to construction. About six families were involved in this work, among them the Monroes, Fogartys and Kennedys. Each group would include a father and sundry sons, with their attendant wives and families.
>
> "These men usually chose the trees and cut their own timber for bridge work. Sawmills were not used, all timber being hewn from logs at the bridge site in order to ensure the best continuity of grain through the timber. One method of gaining bridge timbers from the logs, was the use of pit-saws. Another was the cutting of a V-shaped groove in two logs, forming a trestle, into which a second log is placed. A plumb-bob was set up over the last piece so that straight timbers of the correct size could be hewn from the log. Adze, axe and broadaxe were implements used for this task. The timbers being dealt with were properly seasoned hardwoods, requiring considerable effort and skill in handling.
>
> "When suitable timber was not available at or near the site, it was brought in logs from the nearest source. Bridge carpenters often travelled great distances to find suitable timbers. In western parts of the State, ironbark and tallowwood were scarce and usually had to be brought from the coast, often at great expense by road and rail. The following timber specification is an indication of the types and qualities of timber required:
>
>> *"Timber employed to be tallow-wood for the planking for floor of bridge and culverts; tallow-wood for the ordnance fencing and handrail; approved hardwood for split fencing, and ironbark for the remainder of the work; all to be of the best description, sound, straight, free from sap, wanes, shakes, gum-veins, cores or other defects; to have clean sharp arrises, and to be of the full dimensions shown or specified. Hewn timber to be square, smooth, and free from axe-marks; all sawn timber to be absolutely free from heart. All timber to be barked. With the exception of piles, the diameter given is to be measured at the middle section of the log, exclusive of bark. All timber when delivered at the site to be stacked so as to ensure proper seasoning."*
>
> "Cast-iron chairs [shoes] were often made by the bridge carpenters themselves at the site, from patterns provided by the Department. Buildings and materials handling were carried out in the most elementary manner. Heavy logs and hewn timbers were moved around the site on rollers which were simply a pair of round timbers. Trusses [not Old PWD or McDonald] were usually erected on the approaches to the crossing, and then each truss was launched individually, using a 'gin' pole in the centre of the span, and a second pole on the far abutment. Movement was achieved by hand operated winches."

Figure 7: Construction of Old PWD truss at Clarence Town (R.L. Ford, Williams River, 1995, p 221)

Figure 8: Construction of Allan Truss at Pyrmont (Pyrmont Bridge Conservation Management Plan)

A copy of the original specification for the contract for the construction of a composite truss bridge over the Wakool River at Gee Gee crossing between Swan Hill and Deniliquin (Gee Gee Bridge, a Dare truss) gives insight into the construction of the later truss types (de Burgh and Dare trusses). Some critical information that can be gleaned is that these bridges were constructed on a camber (the steel bottom chord was fabricated with a camber, and the timber was cut to the cambered length), which has implications for appropriate re-cambering techniques. Also of interest is the extent of testing and quality control required for the materials, and the inclusion of some information not included in the drawings (such as washer dimensions).

Following are excerpts from this specification, dated 19 July 1928:

> "This Contract is to include the whole of the work required for the manufacture, supply, and delivery of all materials for, and the construction, erection, and final completion of Composite Truss Bridge with Approach Spans and Approaches over Wakool River at Gee-Gee Crossing, as shown on Plans and described in detail in Specification.
>
> "All trees on the site likely to do any damage to the bridge or approaches by falling or interfering in any way to be cut down stump high, or root-felled, as directed, and removed from the vicinity of the bridge or approaches.
>
> "The Contractor shall supply a moveable wooden office for the sole use of the officer appointed by the Department to superintend the works. Such office shall not be less than 8 ft. x 6 ft. x 8 ft. high, inside measurement, with boarded floor, moveable window sash, and door with lock; to be ventilated and weatherproof, and furnished with approved drawing table, office stool and chair.
>
> "The steel and iron work will be inspected at the Contractor's works during manufacture, but such inspection will not prevent the ultimate rejection of any steel and iron work in which defects or errors may be found previous to the completion of the Contract.
>
> "Any timber which is not thoroughly seasoned at date of inspection must be of such dimensions as to allow the finished sizes, shown on Plans or specified, to be placed in the work after it is seasoned and wrought. The white lead, oil and turpentine used for painting are to be of pure quality, and samples for testing will be taken by the Inspector when the material has been delivered at the site of work, and no painting to be undertaken until materials have been approved.
>
> "Tensile and cold or quenched bend tests will be made on specimens of rolled and forged steel as follows: - For Plates – One each tensile and bend test for each thickness of plates cut both with and across the grain. For rounds and other sections – One each tensile and bend test specimen for each size of bar, except for suspension rods, for which two each tensile and bending test specimens for each size of bar will be required. Bending and flattening tests will be made on not more than 4 per cent. of manufactured rivets.

"The steel to comply with the following requirements: -

Section	Ultimate tensile strength in tons per sq. in. Min.	Max.	Minimum elongation
Plates and all sections other than rounds, under 3/8 in. thick	28	33	16 per cent. in 8 in.
Plates and all sections other than rounds, 3/8 in. and over in thickness	28	33	20 per cent. in 8 in.
Round bars other than rivet steel	28	33	20 per cent. in 8 diameters, or 24 per cent. in 4 diameters.
Round bars for rivets	25	30	25 per cent. in 8 diameters, or 30 per cent. in 4 diameters.

"The iron castings to be of the best grey metal, remelted in a cupola. Castings to be uniform in quality, sound, free from flaws, cracks, sand-holes, air bubbles, scoriae, or other defects, and no stopping, plugging, burning or welding will be allowed, unless with the permission of the Engineer… Castings to be of uniform thickness, with clean and smooth surfaces, and filleted at internal angles, unless shown square: - all runners, seams, &c., to be cut off. Allowance to be made on all castings for contraction and machining, so that they will finish to dimensions shown on Plans.

"Steel bolts – Unless otherwise shown on Plans, all bolts through timber-work to have square heads and nuts, with two square washers to each bolt; bolts through metalwork to have hexagonal heads and nuts, and rounded ends, with one round washer to each bolt under nut; heads to be countersunk, if required; threads to be accurately cut on the bar of the bolt, and screwed to Whitworth's standard; no welding on of screwed ends will be permitted; all nuts to fit hand tight. Unless otherwise shown, bolts to be screwed four diameters in length, and washers for bolts to be as follow: -

Diameter of Bolt.	Square Washers.	Round Washers.
1 1/4 in. ……………………………	3 1/2 in. x 3 1/2 in. x 1/4 in.	2 1/2 in. diameter x 1/4 in.
1 1/8 in. ……………………………	3 1/2 in. x 3 1/2 in. x 1/4 in.	2 1/4 in. diameter x 1/4 in.
1 in ………………………………	3 in. x 3 in. x 1/4 in.	2 in. diameter x 1/4 in.
7/8 in. ……………………………	2 1/2 in. x 2 1/2 in. x 3/16 in.	1 3/4 in. diameter x 3/16 in.
3/4 in. ……………………………	2 1/2 in. x 2 1/2 in. x 3/16 in.	1 1/2 in. diameter x 3/16 in.
5/8 in. ……………………………	2 in. x 2 in. x 1/8 in.	1 1/4 in. diameter x 1/8 in.
1/2 in. ……………………………	1 1/2 in. x 1 1/2 in. x 1/8 in.	1 in. diameter x 1/8 in.

"Timber employed to be tallow-wood, grey box, or ironbark, at the option of Contractor, for the handrails; tallow-wood, ironbark, white mahogany, grey gum, red gum, grey box, blackbutt, or brush box, at option of Contractor, for the planking and kerbs; ironbark, grey gum, red gum, tallow-wood, or grey box for the sheathing and gravel-boards; ironbark, tallow-wood, grey gum, red gum, or grey box for driven piles, and ironbark for the remainder of the work; all to be of approved quality, sound, straight, free from sapwood, large or loose knots, wanes, shakes gum-veins, cores, or other defects; to have clean sharp arrises, and to be of the full dimensions shown or specified.

"Hewn timber to be square, smooth, and free from axe marks, and show no heart on the outside. Sawn timber to be absolutely free from heart, and to be so fixed that the surface which was farthest from the heart of the tree will be the outermost in the work other than planking, and uppermost in the planking.

"Piles of abutments and piers to be driven to the depths shown on Plans, but if at that depth the penetration exceeds 2 inches in piers 3 and 4, 3 inches in piers 1, 2, and 5, and 6 inches in abutments for three blows of a 1 ton ram falling 10 ft. on a perfectly sound pile-head (not brushed) the driving shall, if required by the Engineer, be continued until such test is obtained. If before contract depth is reached the penetration is less than 1/2 in. for three blows, the driving is to be discontinued, if directed.

"Cast iron shoes… Date of casting and letters showing pattern to be cast on respective shoes. Shoes on bottom chords to be carefully bedded in 8-lb sheet lead, and to be a good fit on bearing surfaces, but need not, at the discretion of the Engineer, be machined if castings are true, except upon the butting edges, where they bear against the plates on chord. These edges are to be machined. Shoes B, C, D, to be fitted with two 3/4 in. coach screws, 4 in. long, for securing to top chords. Shoes E, F, and G to be secured to bottom chord each with two 3/4 in. bolts. Shoes A to be secured to braces each with two 7/8 in. bolts. Shoes H to be secured to bottom chord with 7/8 in. and 3/4 in. turned and fitted bolts and fastened to end posts each with two 7/8 in. bolts. Internal angles of all castings to be filleted to a radius of 1/2 in. unless otherwise shown.

"The bottom chords of truss spans cambered 2 in. at centre as shown to be constructed each in three lengths… The chords to be assembled in manufacturer's yard, with windbracing complete, and to be to the satisfaction of the Engineer before forwarding to site. Chords to be packed… to protect them against damage in transit to site.

"Steel rods, 2 in., 1 3/4 in. and 1 1/2 in. diameter, for general lengths, to be arranged as shown on Plan No. 4; the ends of rods to be upset to 2 1/2 in., 2 1/4 in. and 2 in. diameter respectively, and screw cut with Whitworth angular threads. Rods to be fitted with hexagonal nuts and round washers, as shown; no welds will be allowed in rods; the rods to be finished so that screw-cut ends and nuts of a size may be interchangeable without shake. Nuts to be bored out of solid, screw cut and faced.

"Top chords of trusses to consist of two wrought timbers, each 13 in. x 6 in. having saw-fitted butt-joints with steel channel cover-plates, 10 in. x 3 1/2 in. x 5 ft. long, weighing 24.46 lb. per foot, secured with 7/8 in. bolts. Wrought timber packings to be fitted between chord timbers and secured as shown. Weatherings of No. 16 gauge galvanised iron, 27 in. wide, to be secured with 2 in. galvanised lead-headed wire nails over all joints and packing pieces, and at ends of chords where timbers are in contact.

"Side-braces for trusses to be steel T bars, 6 in. x 3 in. x 1/2 in., secured to top chords and cross-girders, with 7/8 in. bolts. Those at hip joints to be set to clear hip castings…

2.4 Timber Truss Bridge Type 1: Old PWD Truss

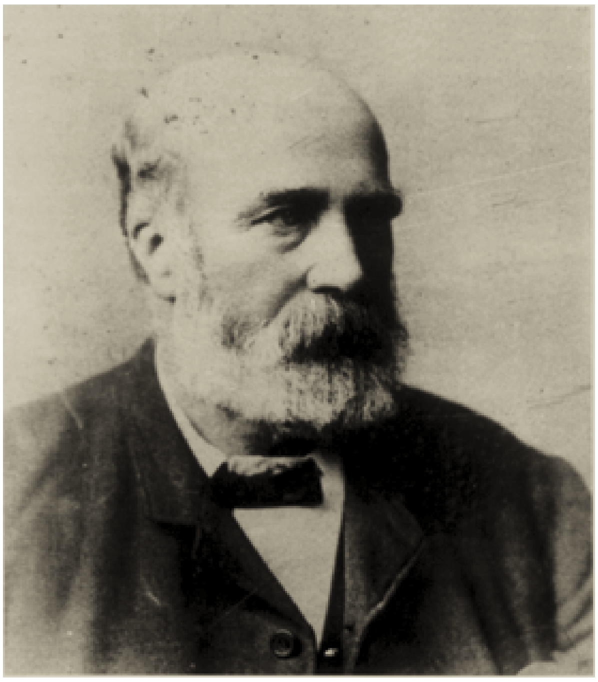


Figure 9: William Christopher Bennet

Figure 10: An "Old PWD" Truss Bridge

William Christopher Bennett was born in Ireland on the 4th of July 1824. After being employed on territorial and railway surveys and drainage works in Ireland, by the age of twenty two he had four or five thousand men under him and was acting as District Engineer. He then worked in South America, England and New Zealand, as well as visiting France before coming to Australia.

Bennett was appointed Assistant City Engineer in New South Wales in 1855 until 1856. He worked under John Whitton, the Engineer-in-Chief for Railways from 1857 until 1858 when he was selected by Captain Martindale, Commissioner for Internal Communication, to superintend the repair of a flood damaged bridge at Bathurst. Martindale was so pleased with the manner in which this work was completed that he recommended Bennett for the position of Engineer to the Roads Department, to which he was appointed in Jan 1859 to Jan 1861. After an absence of twelve months, in which he returned to England, he worked again in the Railway Department under Whitton before receiving the appointment of Commissioner and Chief Engineer to the Roads Department, which he retained until his retirement from the public service on 1 July 1889.

Up to the end of 1888, the total length of main roads, metalled and gravelled, was nearly 6,000 miles, in addition to nearly 4,000 miles of unmetalled roads. About 40 miles of bridges had been constructed, many of them the largest in the southern hemisphere. Bennett was described as, "a most conscientious and upright man, an energetic worker, a strict disciplinarian in his department, and fearless and impartial in the administration of his public duties." To Mr Bennett is generally accorded the reputation of being one of the ablest engineers in Australia, and many of his works (including two timber truss bridges) stand as monuments of his ability and great labour.

Bennett was a man of courage, as can be seen from the following excerpt from his obituary:

> *"Mr Bennett executed fully all the surveys and explorations entrusted to him, surveying and levelling by himself a large tract of country towards the Chuqunaque River [Panama]; having no companion through that hostile country but black chainmen. He also assisted to bury some men belonging to H.M.S. "Virago" under the command of Captain Prevost, who were shot by the Indians while he was there; and afterwards accompanied Lieutenant Forsyth… for the rescue of Lieutenant Strain, of the US Navy, and his missing party, in which they succeeded; and for this service, Mr Bennett received the thanks of the American Government through the Secretary of the United States Navy…"*

Bennett was well loved and respected by those who worked for him, as can be seen by the collection of his letters kept at the Mitchell Library. A former colleague of Bennett wrote:

> *"Our late chief, Mr W.C. Bennett… was a man of singular ability, prodigious energy, and untiring industry… The immense department which has grown up under Mr Bennett's control, and the work it has done, will probably not be chronicled till it, like he, has broken down under the strain, increasing as it does from year to year. Both have done their work nobly and well; both deserve the honour not always accorded where most merited."*

In addition to his prodigious work on roads and bridges in New South Wales, Bennett also made a significant contribution to navigation, water supply and sewerage works. In 1852 he accepted appointment with Gisborne & Forde to go from Ireland to South America and report on the navigation of the Magdalena River, its connection with the sea by canal and the possibilities of a further canal link with Bogota, capital of Nueva Granada (Colombia). As a preliminary he toured the Rhone and Saône Rivers in France to study methods of river navigation by large boats. After he returned to England from Colombia, he helped to plan a proposed embankment for the Thames, which was, however, never implemented. In 1853 he re-joined Gisborne & Forde in another expedition to Latin America, this time in charge of surveying and exploring the Pacific side of the Isthmus of Darien for the international ship canal. It was there that he also assisted Lieutenant Forsythe and a detail from H.M.S. Virago in the hazardous rescue of a missing exploration party of United States navy personnel under Lieutenant Strain. Interestingly, this expedition was written up, illustrated and appeared over three successive editions of the 1855 Harper's New Monthly, a periodical of the day, with glowing reports of Bennett's contributions.

> *"The noble-hearted Bennett… a stranger and foreigner - this grand, high purpose to cast his lot in with the distressed commander, and save his party, or perish with them - reveals one of those lofty, elevated characters which shed lustre on the race."*

Letters and testimonials from his superiors, subordinates and friends indicate that Bennett had great ability both as an engineer and as an administrator. Ambitious in the tasks he was prepared to undertake, he drove his subordinates hard but was loyal and generous in return and made staunch friends among them. In 1872 Sir Henry Parkes, speaking in support of an increase in Bennett's salary, described him in parliament as, "one of the ablest officers in the government service" and asserted that he had been grossly underpaid for his important and competent work.

There has been much written about the influence of American and British bridge building technology and the influence on the early bridge designers in New South Wales. It seems that it is sometimes overlooked that Bennett designed an extraordinary variety of timber bridges.

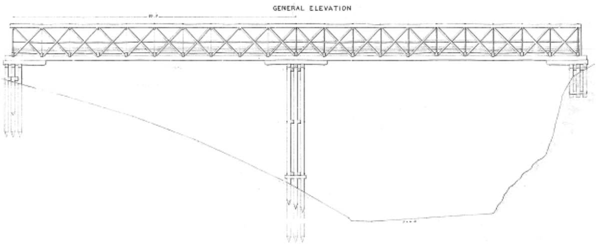


Figure 11: Bennett's Vacy Bridge, 1858 (83' 9")

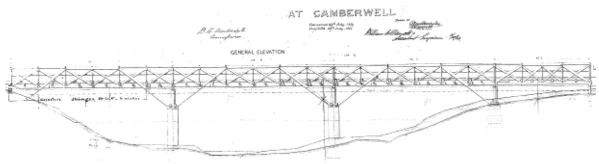


Figure 12: Bennett's Falbrook Bridge, 1858 (68½')

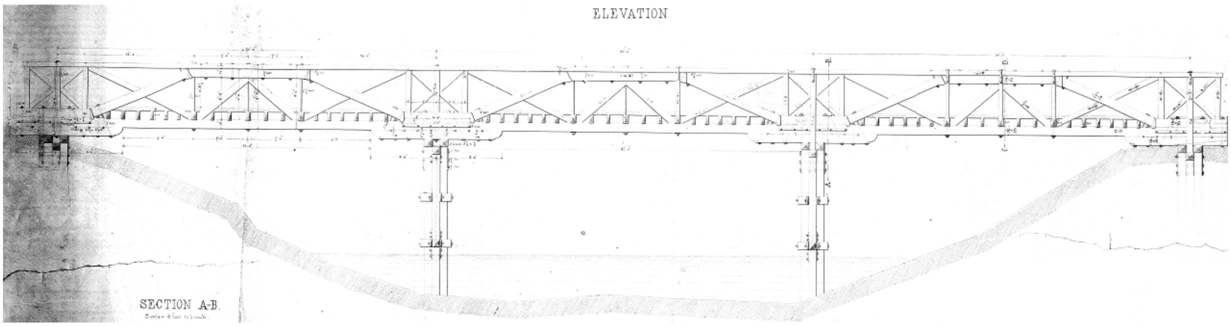


Figure 13: Bennett's Camden Bridge, 1860 (50' Spans)

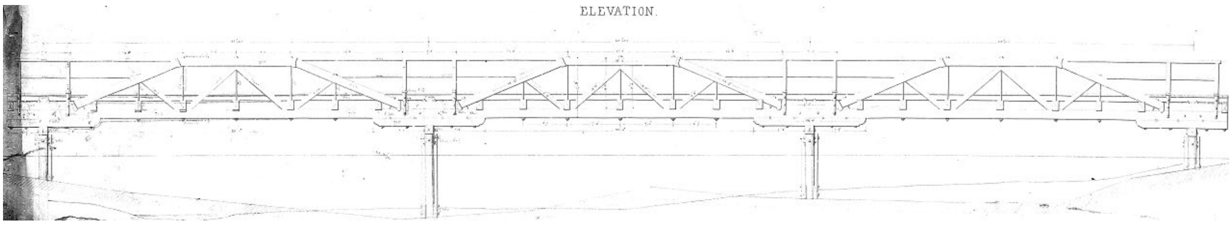


Figure 14: Bennett's Murrurundi Bridge, 1860 (50' Spans)

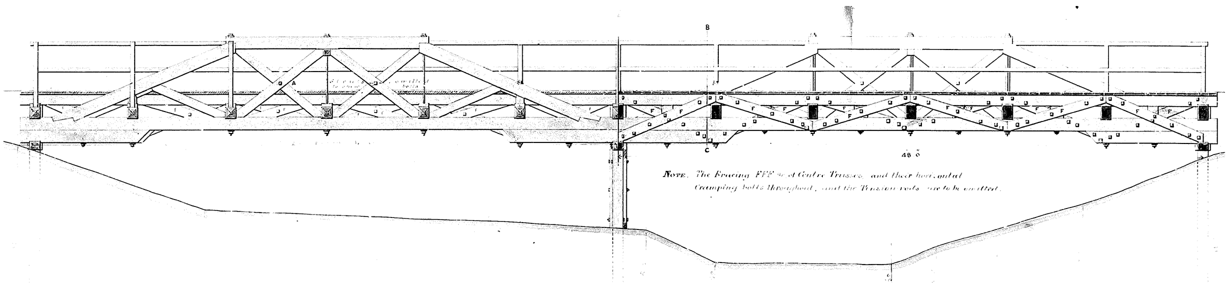


Figure 15: Bennett's Rope's Creek Bridge, 1860 (48' Spans)

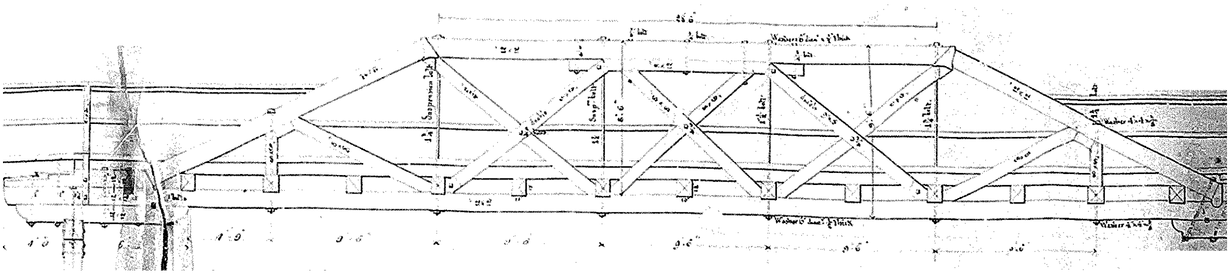


Figure 16: Bennett's Christmas Creek Bridge, 1865 (70' Spans)

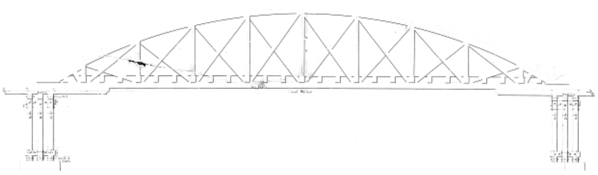


Figure 17: Bennett's Bendemeer Bridge, 1868 (102')

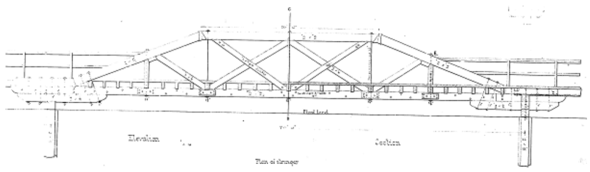


Figure 18: Bennett's Reedy Ck Bridge, 1869 (70')

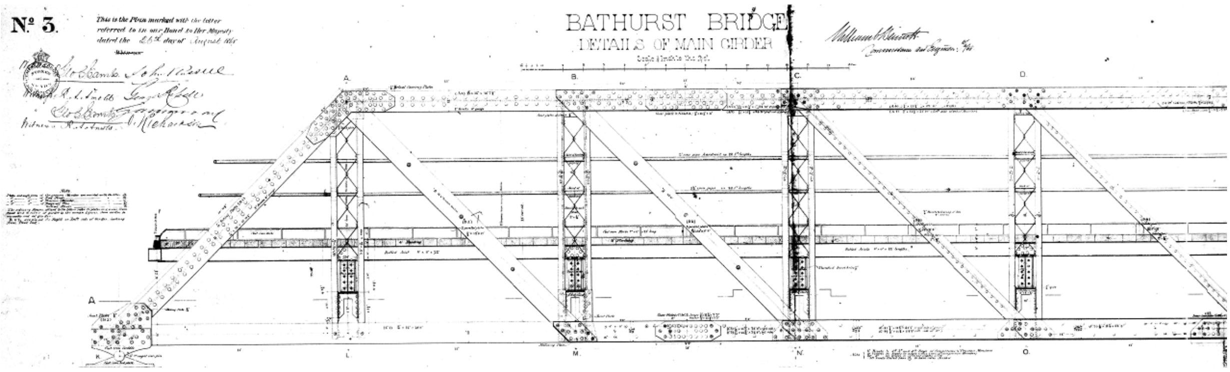


Figure 19: Bennett's Iron Bridge over the Macquarie River at Bathurst (Denison Bridge), 1868 (113'-Iron)

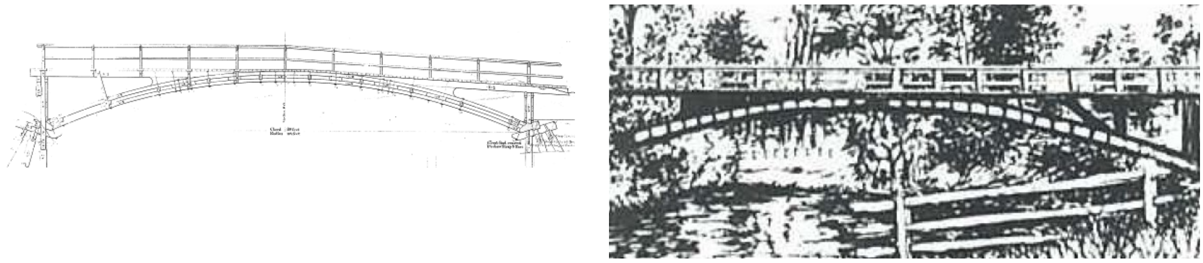


Figure 20: Bennett's Brougton Ck Bridge (plan on left, sketch on right), 1870 (80')

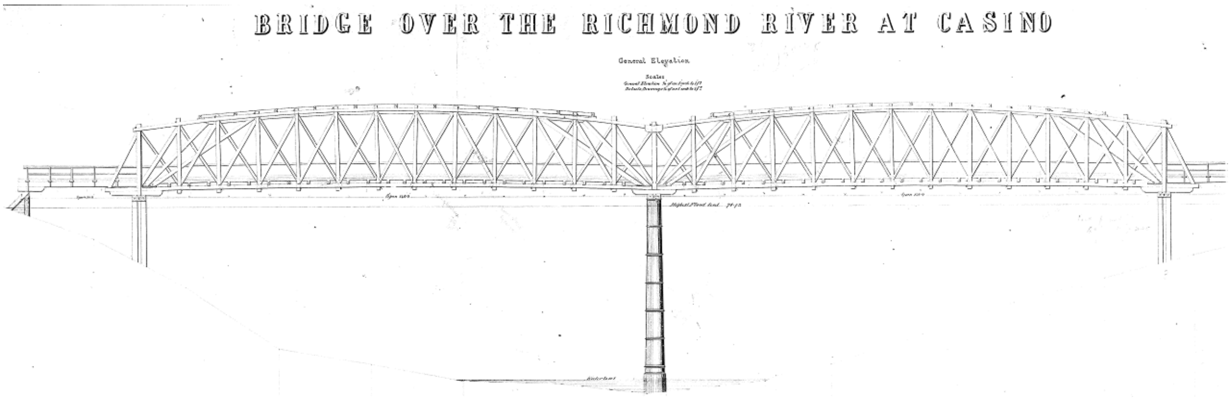


Figure 21: Richmond River Bridge, 1874 (140') (not signed by Bennett, but mentioned in his report)

One of the first bridge designs signed off by William C. Bennett in New South Wales (Vacy Bridge, 1858) is almost indistinguishable from an American timber Howe Truss design. It was a two span continuous timber truss bridge with laminated timber top and bottom chords. Cross girders were located only at panel points, and the width of the carriageway was only 12 foot. Each span was approximately 25 m in length, making the total truss length more than 50 m. It was a high level bridge, considerably elevated above the Paterson River which it crossed.

Another bridge design signed off by Bennett very early in the development of timber truss bridges was the bridge over Falbrook at Camberwell. This bridge shares some details with Vacy Bridge, in that its basic structure looks similar to a Howe Truss, however, Bennett appears to have made use of splice plates and scarf joints in the top and bottom chords rather than providing laminated timbers. Also, the spacing of cross girders is reduced so that an additional cross girder is supplied midway between panel points in addition to cross girders at panel points. As well as these changes, radiating principals were adopted on the longer spans which extend well below the level of the bottom chord of the truss, and connect to the piers. The splayed principals appear similar in concept to those introduced many years later in the McDonald truss, with two flitches and timber spacers held in position with bolts. The bridge consisted of four spans (3 x 68’ 6” + 1 x 48’ 6”) with the truss being continuous over the whole 75 m length, and the carriageway width being 12 foot. An interesting thing to note in the bridge at Camberwell is the thickened top chords for the central panels, between the points where the splayed principals meet the top chord. In fact the shorter span includes all the elements of an early “Old PWD” type truss, with additional vertical members at the ends and an extended top chord. Not all of the early “Old PWD” type trusses made use of cast iron shoes, although cast iron shoes were introduced in at least one “Old PWD” type truss design as early as 1860. Another important difference between the bridge at Vacy and the bridge at Camberwell is the lack of corbels at Camberwell, which again, is usually only a feature in McDonald type timber truss bridges.

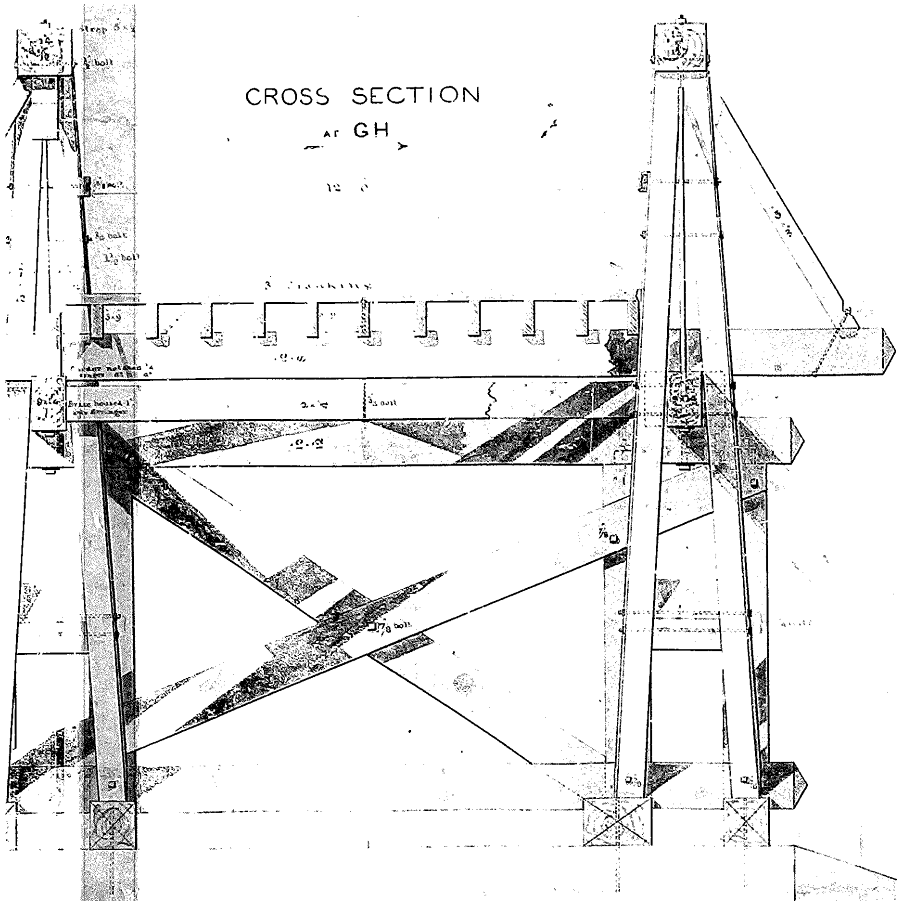


Figure 22: Cross Section of Bridge over Falbrook at Camberwell Showing Long Splayed Principals (1858)

It is clear that Bennett was generally aware and comfortable with American practice in bridge building, especially since one of his very first bridges was an American Howe Truss. However, it is also important to note the British influence, as Bennett did often choose to make use of the queen truss. One of the first bridges he proposed in 1858 was a timber queen post truss for Farmers (or Junction) Creek on the Western Road, shown in the figure below. One of the interesting aspects of this design was that it included no cross girders, but thick transverse decking that spanned between the trusses. His use of corbels under the truss span as well as attention to detail at the connections separates Bennett's design from earlier designs by the Colonial Architect's Branch for a bridge over Paddy's River, which was opened in 1855, and consisted mostly of a timber trestle bridge, but with one king bolt and one queen post truss span.

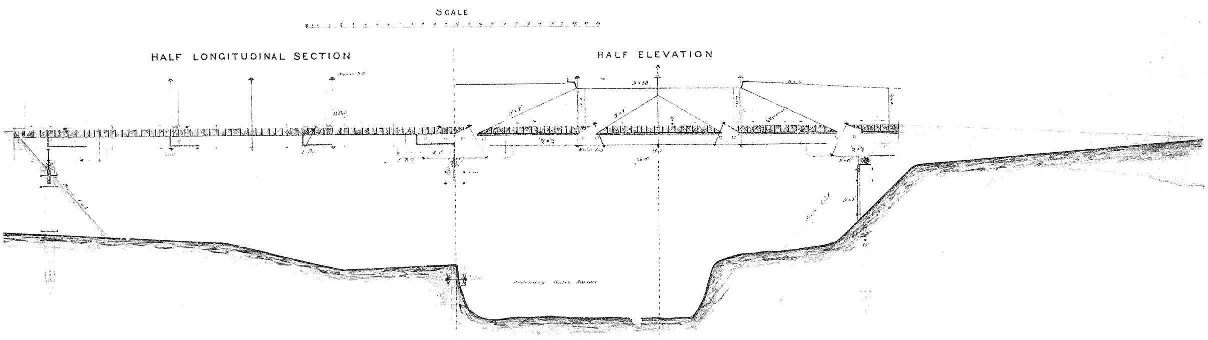


Figure 23: Bennett's Bridge over Farmers (or Junction) Creek, Western Road, 1858 (approx 30' spans)

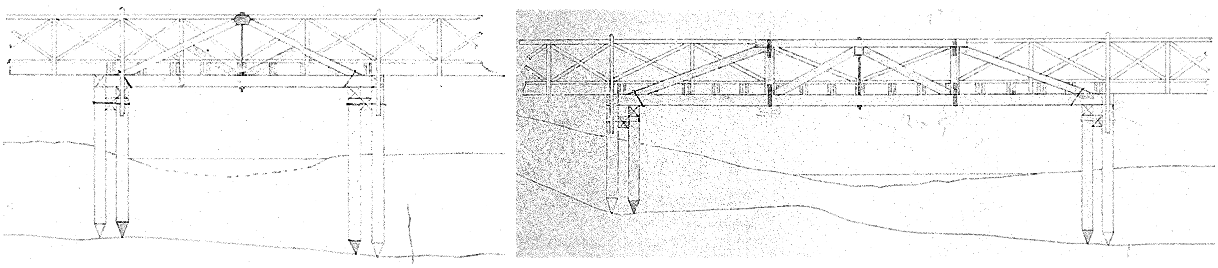


Figure 24: Colonial Architect's King Bolt (left) and Queen Post (right) trusses for Paddy's River (1855)

A report written by Bennett dated 31 March 1865, and published in the Sydney Morning Herald states: "*The first bridges constructed by the department were found to be narrow in the road way; now a minimum width of 18 feet has been adopted for bridges in and near towns, and from 14 to 16 feet in the more remote localities; spans of simple beams with corbels have been used up to 35 feet and are now being constructed to 40 feet. For trussed bridges, the simple queen truss with iron suspension rods in spans of from 50 to 90 feet has been used, as giving the greatest headway and requiring least workmanship. When the headway has not been limited, a modification of this truss with radiating principals has been adopted, with the tie beam passing between the principals; it has been used in spans of from 60 to 100 feet, and the laminated arch has been applied in spans of the same dimensions, in some special cases where timber large enough for trusses could not be obtained. As yet, from want of full experience of the capabilities of the indigenous timber applied to intricate framing, and from the very shrinkage and warping which occurs if not seasoned, spans exceeding 100 feet have not been used; but a design for large spans, on the principal of the McCallum truss, so extensively used with the softer and lighter timber in the United States, has been under consideration for some time, and will be applied when opportunity offers.*"

2.4.1 Characteristics of the Old PWD Type Timber Truss Bridges

The primary characteristics that distinguish William Christopher Bennett's designs for the Old PWD type timber truss bridges from other timber truss bridges are as follows:

- The top chords and end principals consist of large section solid sawn timber members.
- Principals are significantly longer than diagonals, with a vertical timber prop approximately half way along the length and a vertical tension rod also at that location. A timber spacer separates the vertical timber prop from the timber diagonal also supporting the principal.
- Principals are supported at the base on long timber butting blocks. Butting blocks are bolted to the bottom chord, and timber shear keys or notches are used to transfer the loads. There is a tear-drop shaped cast iron shoe provided between the principal and the butting block.
- Timber sway braces are provided at all central panel points to laterally support the top chord.
- Bottom chords consist of three sawn timber laminates bolted together to form the same section size as the top chord and principal. Joints for all laminates occur at panel points, and small metal splice plates are provided at each joint. Bottom chords are continuous over piers.
- Spans have two, three or sometimes four counterbraced central panels between principals.
- For trusses with three central panels, the middle panel has a double top chord, and for trusses with four central panels, the middle two panels have a double top chord. Longer spans (e.g. Clarence Town) have the double top chord extending beyond the panel points, whereas others (e.g. Monkerai Bridge) have the double top chord stopping neatly at the panel points.
- Cross girders are generally (but not always) closely spaced, and carry diagonal decking.
- Either single or double vertical wrought iron tension rods are installed through holes drilled in the top and bottom chords. Larger diameter tension rods are generally provided towards the ends of the top chords where stresses are higher than the smaller tension rods towards the centre of the span. In the case of designs with four central panels, the middle tension rod is a single member whereas all the other tension rods are doubled.
- Railings consist of simple single (on smaller trusses such as Monkerai) or double (on larger trusses such as Clarence Town) rectangular timber rail, attached directly to the truss (no vertical posts). There were no kerbs provided, and there were no enlarged timber end posts.

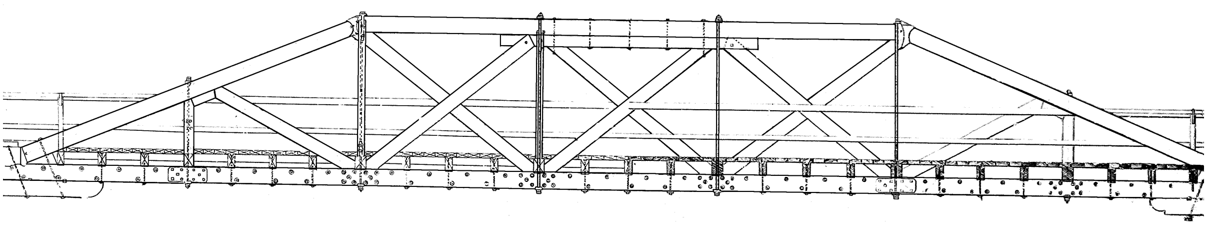


Figure 25: Clarence Town Bridge, Old PWD, Designed 1878, Two Truss Spans of 100 ft each.

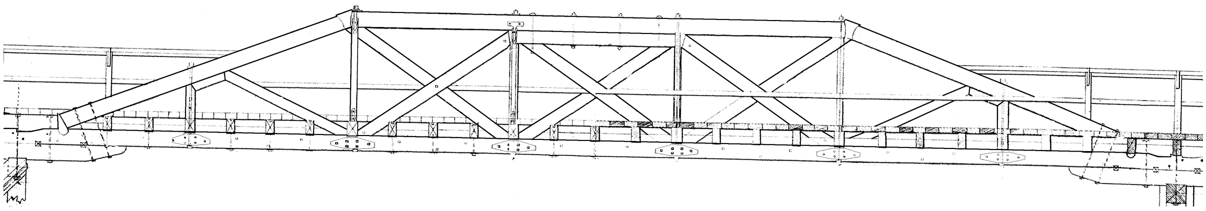


Figure 26: Monkerai Bridge, Old PWD, Designed 1881, Three Truss Spans of 70 ft each.

2.4.1.1 Conserving Engineering Heritage in Old PWD Type Timber Truss Bridges

According to the NSW Heritage Division, "the main aim in assessing significance is to produce a succinct statement of significance, which summarises an item's heritage values. The statement is the basis for policies and management structures that will affect the item's future."

Items listed on the RMS Section 170 Heritage and Conservation Register or on the NSW Heritage Division's State Heritage Register will generally have a Statement of Significance which should inform the design process. The Statement of Significance should include not only technical significance, but also historical, associational, aesthetic, social, archaeological, rarity and representativeness where relevant. However, since these structures are engineering heritage, a Statement of Significance for an Old PWD truss bridge would rightly include the following:

> *As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and with William Christopher Bennett, the Commissioner for Roads, one of Australia's ablest engineers, the designer of this truss type. Old PWD trusses were the first in the five-stage development of NSW timber truss road bridges. The trusses took advantage of the high quality hardwood that was plentiful in NSW. The design is an example of innovative and practical engineering in a time when large and long section timbers were readily available and vast numbers of bridges were being built, but budgets were tight and skilled workmen were few.*

The Statement of Significance must inform the design process. Examples of how the Statement of Significance above should inform the conservation of Old PWD trusses are given here.

The historical context of these bridges is plentiful quality hardwood, particularly that large and long section timbers were readily available and vast numbers of bridges were being built, but budgets were tight and skilled workmen were few. The conservation of these bridges should not obscure this important context, that is, the use of long and large section NSW hardwood timbers.

These bridges have strong associations with the expansion of the road network and economic activity throughout NSW. Therefore, the conservation of these bridges should retain their use as a vital part of the NSW road infrastructure, which will generally require their strengthening.

These bridges also have strong associations with William Christopher Bennett. These bridges have the opportunity to demonstrate the engineered design details of this type of truss. Although many of the details may have changed throughout the life of the bridge, there is generally sufficient evidence of the original design both in drawings and in old photographs to allow restoration and reconstruction within the bounds of Articles 19 and 20 of the Burra Charter. Conservation of these bridges should involve returning to original design details where possible.

As timber truss bridges, Old PWD trusses are aesthetically distinctive and have landmark qualities. However, it is the innovative and practical engineering which is particularly notable, and this engineering excellence is seen, not primarily in the general shape of the trusses, but in their details, the flow of forces, the connections and the structural rigidity. Therefore, the conservation of these bridges should not obscure the original design intent in the design details.

2.4.1.2 Remaining Old PWD Type Timber Truss Bridges and Listings (as of 2013)

Of approximately 150 Old PWD type timber trusses built in New South Wales between 1858 and 1886, two remained in 2013. These are listed below. Both are the responsibility of RMS.

Table 1: Heritage Listings for Old PWD Type Timber Truss Bridges			
	SHR	Section 170	LEP
RMS Bridges			
Clarence Town Bridge over the Williams River	Yes	Yes	Dungog
Monkerai Bridge over the Karuah River	Yes	Yes	Not listed

2.4.1.3 Gradings of Significance in Old PWD Type Timber Truss Bridges

The method for grading significance given by the NSW Heritage Division is heavily influenced by the presence or absence of original fabric. This can be problematic for exposed timber structures such as bridges, in which none of the timber is likely to be original fabric, but timber elements are replaced cyclically and completely during the life of the structure. The longitudinal timber sheeting (not an original design detail, but necessary for modern vehicular loads) lasts approximately seven years before having to be replaced with new timbers. The transverse or diagonal timber decking lasts up to 15 years before having to be replaced with new timbers. Round timber girders frequently found on approach spans on average last 30 years. Truss members such as diagonals on an Allan or Dare truss can last up to 50 years. Timber piles are also very susceptible to rot and termite attack in the region just below the ground surface, so installation of new piles is a critical and regular aspect of bridge maintenance.

Unfortunately, timber is not the only material that is unlikely to be original in a timber truss bridge. Original wrought iron tension rods in Old PWD type trusses have very often been replaced with larger steel tension rods in order to facilitate maintenance activities such as cambering. The original cast iron shoes in all truss types are very susceptible to brittle fracture, and have often been replaced either with new cast iron shoes, or sometimes with welded steel shoes. Cast iron piers are generally original, but are very susceptible to graphitisation, which has lead to considerable section loss in many of piers of the remaining timber truss bridges.

The ICOMOS principles for timber structures recognise the vulnerability of timber structures to material decay and degradation, and so the primary aim of preservation and conservation is, "to maintain the historical authenticity and integrity of the cultural heritage" rather than the original fabric per se. Similarly, "the aim of restoration is to conserve the historic structure and its load bearing function and to reveal its cultural values by improving the legibility of its historical integrity, its earlier state and design…" It is appropriate, therefore, to allow the ICOMOS principles to guide the gradings of significance in addition to the Heritage Division guidelines.

Table 2 provides generic gradings of significance for each of the primary characteristics of an Old PWD truss, along with some implications for conservation of this truss type.

Table 2: Gradings of Significance for Old PWD trusses with Implications for Conservation

	Primary Characteristic	Implications for Conservation
Exceptional	Principals are significantly longer than diagonals, with vertical and diagonal timber props approximately half way along the length with additional tension rods.	This relates to the shape of the truss, and is one way it can be recognised as an Old PWD truss. It should not be modified.
	For trusses with three central panels, the middle panel has a double top chord. Longer spans have the double top chord extending beyond panel points. Shorter spans have the double top chord stopping at panel points.	This relates to the shape of the truss, and is one way it can be recognised as an Old PWD truss. It should not be modified.
	Designs have two, three or sometimes four central panels between principals. Central panels generally (but not always) have counterbracing as well as bracing.	This relates to the shape of the truss, and is one way it can be recognised as an Old PWD truss. It should not be modified.
	The top chords and principals consist of large section solid sawn timber members.	Alteration detracts from significance by obscuring the reason for this truss' existence (plentiful long and large section timbers). Also, use of smaller members bolted together to form larger members significantly reduces the strength of the bridge.
	Iron castings (also called shoes) are provided at each end of principals, the bottom one is tear-drop shaped.	The flow of forces at the castings of the Old PWD trusses is unique and essential to the original design intent. Although the shape of the top shoe changed, the bottom shoe was unchanged.
High	Bottom chords consist of three lines of sawn timber laminates bolted together to form the same size as the top chords and principals, thereby framing the truss. Joints for laminates occur at panel points, and small metal fish plates (sometimes called splice plates) are provided at each joint.	Increasing the size of the top chord, the principal or the bottom chord impacts on the design of the truss, which always had these three members of equal dimensions. Original lengths of timber used in the bottom chord are no longer available, but shorter lengths significantly reduce the strength of the bridge, and so some form of strengthening by introduction of other materials is necessary.
	Sway braces are timber, detailed to give lateral stability to the tops chord.	The purpose of the original sway bracing to provide lateral stability to the top chord is unique and essential to the original design intent. With timber cross girders being replaced with steel due to heavier loads, it may not be possible to retain the timber design, but sway braces should provide lateral restraint and be clearly distinguishable from sway bracing in later truss types.
	Either single or double vertical tension rods are installed through holes drilled in the top and bottom chords. The rods at the ends of top chords are larger than others.	The original wrought iron tension rods are subject to failure due to deterioration and increased loads, and so these elements are generally replaced with steel tension rods having a greater diameter. It is important that the size of the tension rods reflect the flow of forces, so larger tension rods should be supplied at the ends of the top chord, and smaller tension rods towards the centre of the span.
Moderate	Timber cross girders are generally closely spaced, and carry diagonal decking.	These elements contribute to the overall significance of the bridge, but are not essential or unique to this truss type, nor are they the only decking system that was used for this truss type. Other deck systems may assist in conservation.
	Butting blocks bear against outer approach span girders continuous back to abutments.	This detail contributes to the overall understanding of the design, which provided a number of redundant load paths for increased structural reliability. However, if new materials (such as steel) are introduced to strengthen the bottom chord, then thermal expansions and contractions of the steel will make this feature unworkable.

2.5 Timber Truss Bridge Type 2: McDonald Truss

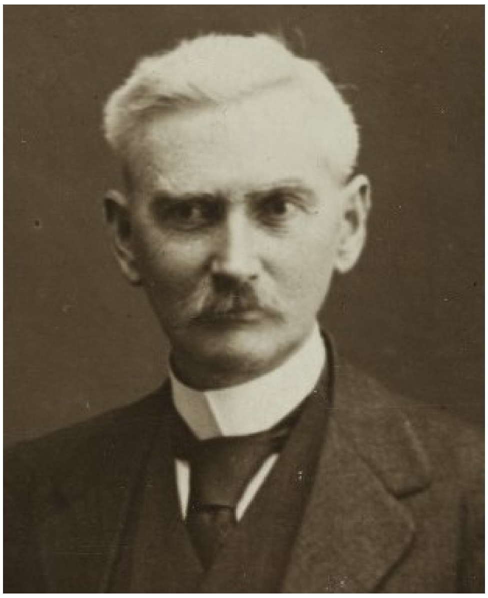


Figure 27: John Alexander McDonald

Figure 28: A "McDonald" Truss Bridge

John Alexander McDonald (more often known as John A. McDonald) was a member of the Institution of Civil Engineers, London, of the Institution of Mechanical Engineers, London; and of the American Society of Civil Engineers, New York. Born on 10 January 1856, he studied at King's College, London, and after working for some time in England, he migrated to New South Wales in 1879 to superintend the erection of the iron bridges over the Parramatta River and Lane Cove, two of the largest bridges that time undertaken by the New South Wales Government. Upon the death of Bennett in 1889, McDonald was appointed Engineer for Bridges for New South Wales. After 14 years service he was retrenched (probably due to the economic depression in Australia in the 1890s). He then spent six months in the United States before returning to take up a position as Assistant Engineer to the Public Works Department in Western Australia and then Resident Engineer for Freemantle Harbour Works. In 1898 McDonald moved to South Africa, where he engaged in mining, designing electric light stations and various other activities until 1908, when he returned to Australia and tried his hand at farming. In 1912 McDonald moved to Gisborne, New Zealand and was Harbour and Port Engineer until 1917, then Gisborne Borough Engineer from 1918 to 1924. He was the designer of the Gisborne Peel Street and Gladstone Road bridges. He was a chronic sufferer from insomnia in his latter years, and he died by his own hand (he shot himself in the head) on 4 June 1930.

He was the patentee of "McDonald's patent expansion rollers" for large bridges, used extensively in New South Wales, Queensland and America, and for which he received awards and medals at London, Chicago, Adelaide and Melbourne. One of the Chief Engineers that worked with McDonald in South Africa furnished a testimonial to the valuable assistance he had received from Mr McDonald, and to his high professional qualifications as well as to the judgment, tact, and courtesy which had always been conspicuous features of his administration.

McDonald made a significant contribution to bridge design in NSW. His achievements include redesign in 1886 of the standard PWD truss to produce the McDonald truss; design of all the lattice girder road bridges from 1884 to 1893, design of a series of sliding bridges at Lismore, Coopernook and Erina; and design of early bascule and lift bridges such as over the Darling River at Bourke. It should not be overlooked, however, that there were two other engineers that joined the Roads and Bridges Branch at a similar time, and undoubtedly assisted greatly. These two engineers are Percy Allan, who joined in 1878, and William Henry Warren, who worked in the Roads and Bridges Branch in 1881 before taking up a position at the University of Sydney.

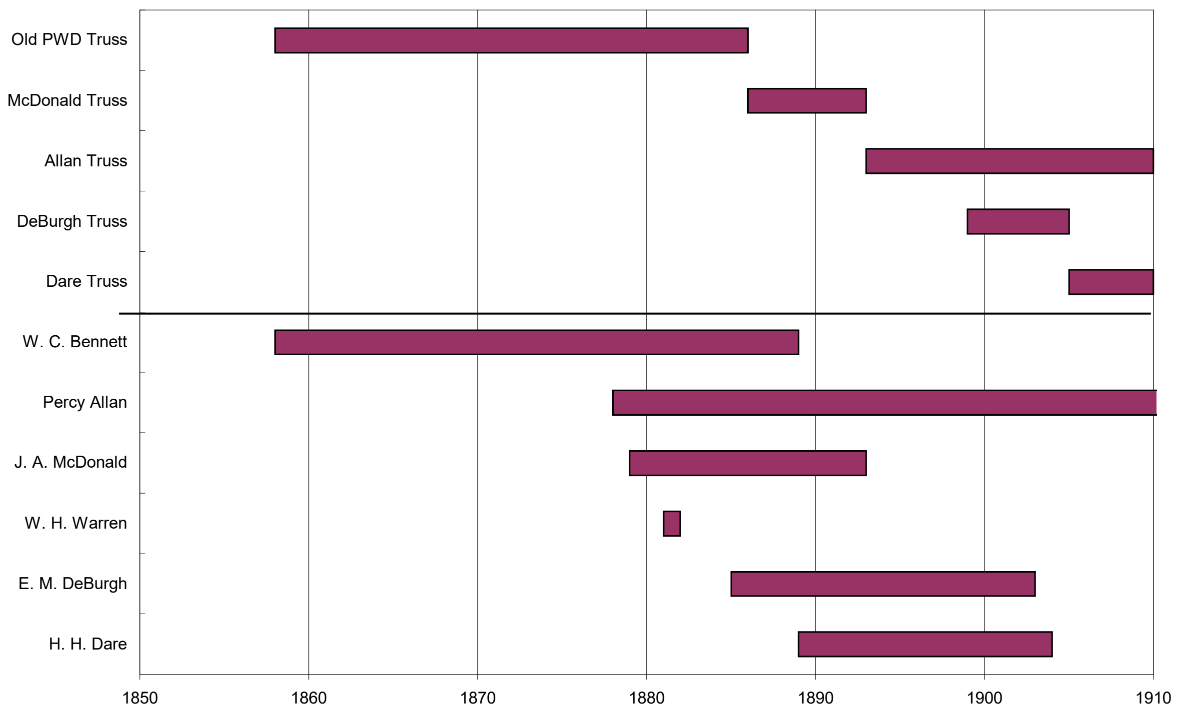


Figure 29: Years Employed by PWD Roads & Bridges Branch and Years Truss Types were Constructed

William Henry Warren was born in England in 1852 and migrated to Sydney in 1881 after working for the London Railways. In Sydney he worked in the Roads and Bridges Branch of the PWD while teaching applied mechanics at Sydney Technical College in the evenings. In 1882 he was appointed the first lecturer in engineering at the University of Sydney. An acknowledged leader of his profession, with a reputation extending beyond Australia, in 1919 Warren was the unanimous choice as first president of the new Institution of Engineers, Australia. Possessing clear insight, depth of knowledge, wide experience and mental ability, he had the gift of delegating work and authority. One of his major achievements was to convince the engineering industry by his personal example that graduate engineers were a sound investment.

From his time working for the PWD, Warren noted the impressive range of hardwood timbers native to eastern Australia, as well as the fact that data on the properties of this resource was difficult to find in that no Australian testing facility of any reliability was available. At the University of Sydney, Warren obtained a Greenwood & Batley Testing Machine in 1886.

Figure 30: Greenwood and Batley Testing Machine at the University of Sydney

On the 1st December 1886, Warren was able to report to the Royal Society of New South Wales the results of strength and elasticity of ironbark timber obtained. John A. McDonald had been a collaborator in the tests and Warren gives him credit for the invention of an autographic strain recording device for the tests. Another important engineer who assisted in many of Professor Warren's tests was Henry Harvey Dare, who was a student at the time. McDonald commented in 1886 that Professor Warren's testing had provided valuable and interesting data, "which every engineer in this colony has felt the need of and been unable to obtain with accuracy."

This new data enabled McDonald immediately to modify the standard design to accommodate higher vehicular loads, as described by Percy Allan, commenting on the Old PWD type trusses:

"Although this type of truss has for many years carried the traffic without accident, yet in view of the increase in settlement and the greater risk of the structures being subjected to heavier loads, it was thought desirable in 1886 to adopt [the McDonald truss design]… These structures were designed for a distributed live load of 84 lbs. per square foot of roadway and a traction engine weighing 16 tons, on a 10'4" wheel base having 9½ tons on the leading wheels."

McDonald also pioneered the new technology of composite trusses where timber and metal are used to their best purposes (metal in tension, timber in compression). In the bridge over the Lachlan River at Cowra in 1893 he demonstrated the potential for composite construction with a span of 160 feet, only exceeded by the 165 foot de Burgh truss over the Lane Cove River in 1900.

2.5.1 Characteristics of the McDonald Type Timber Truss Bridges

The primary characteristics that distinguish John A. McDonald's designs for the McDonald type timber truss bridges from other timber truss bridges are as follows:

- Principals are significantly longer than diagonals and are splayed at the base in order to provide lateral stability to the top chord (which is not provided by the iron sway braces), with a vertical tension rod and a timber diagonal approximately half way along the length.
- Principals are supported at the base on long timber butting blocks (similar to the Old PWD truss). Butting blocks are bolted to the bottom chord, and timber notches are used to transfer the load. There is no shoe provided between the principal and the butting block.
- The bottom chords are constructed from four sawn timber laminates bolted together. Butt joints for internal laminates occur towards the ends of the span and have no metal splice plates. Butt joints for external laminates occur toward the centre of the span and there is a large splice plate provided on both sides of the bottom chord to cover both butt joints. Bottom chords are designed continuous over piers, with no corbels on truss spans.
- Vertical tension rods at each end of the top chord consist of two rods placed outside the chord and held in place by cast iron cradles. All other tension rods are single rods installed through holes drilled into the top and bottom chords (similar to the Old PWD truss).
- Top chords consist of single large section timbers (similar to the Old PWD truss).
- Iron wedges are provided to take up the slack at the base of all bracing and counterbracing.
- Cross girders are closely spaced, with primary cross girders at panel points, and three intermediate cross girders in between. Diagonal decking is used and kerbs are provided.
- Double members in central panels have timber spacers at each end, with grain of timber filling pieces (spacers) generally running in direction of the member. Spacers are also provided in the principals and here spacers are structural and are notched into flitches.
- Iron T sections (6" x 4" x ½") are used for sway braces which are provided at all top chord panel points excluding each end of the top chord, where no sway bracing is provided.

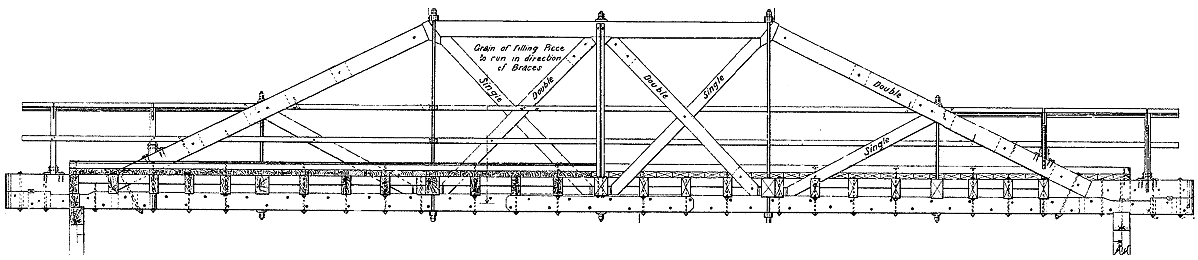


Figure 31: Galston George, McDonald Truss Designed 1892, Single Span 65 ft.

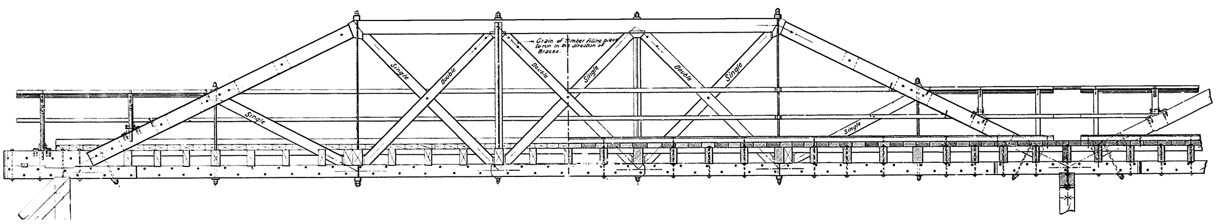


Figure 32: Junction Bridge over Tumut River, McDonald Truss Designed 1892, Three 75 ft Spans.

2.5.1.1 Conserving Engineering Heritage in Old PWD Type Timber Truss Bridges

According to the NSW Heritage Division, "the main aim in assessing significance is to produce a succinct statement of significance, which summarises an item's heritage values. The statement is the basis for policies and management structures that will affect the item's future."

Items listed on the RMS Section 170 Heritage and Conservation Register or on the NSW Heritage Division's State Heritage Register will generally have a Statement of Significance which should inform the design process. The Statement of Significance should include not only technical significance, but also historical, associational, aesthetic, social, archaeological, rarity and representativeness where relevant. However, since these structures are engineering heritage, a Statement of Significance for a McDonald truss bridge would rightly include the following:

> *As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and with John A. McDonald, then Engineer for Bridges, a very capable engineer, and the designer of this truss type. McDonald trusses were the second in the five-stage development of NSW timber truss road bridges. The trusses took advantage of the high quality hardwood that was plentiful in NSW. The design is an example of practical engineering in a time when budgets were tight. The evolution in design shows the growing knowledge of timber as a structural material, and also the increasing vehicle weights requiring stronger bridges than before.*

The Statement of Significance must inform the design process. Examples of how the Statement of Significance above should inform the conservation of McDonald trusses are given here.

The historical context of these bridges is plentiful quality hardwood and so their conservation should not obscure this important context of long and large section NSW hardwood timbers.

These bridges have strong associations with the expansion of the road network and economic activity throughout NSW. Therefore, the conservation of these bridges should retain their use as a vital part of the NSW road infrastructure, which will generally require their strengthening.

These bridges also have strong associations with John A. McDonald, and have the opportunity to demonstrate the engineered design details. Although many of the details may have changed throughout the life of the bridge, there is generally sufficient evidence of the original design both in drawings and in old photographs to allow restoration and reconstruction within the bounds of Articles 19 and 20 of the Burra Charter. Therefore, the conservation of these bridges should seek to apply engineering excellence so as not to obscure the work of this very capable engineer.

As timber truss bridges, McDonald trusses are aesthetically distinctive and have landmark qualities. However, it is the growing knowledge of timber as a structural material which is particularly notable, and this scientific approach to bridge design is seen, not primarily in the general shape of the trusses, but in their details. Therefore, the conservation of these bridges should not obscure the original design intent in the design details.

2.5.1.2 Remaining McDonald Type Timber Truss Bridges and Listings (as of 2013)

Of approximately 90 McDonald type timber trusses built in New South Wales between 1886 and 1893, five remained in 2013. These are listed below. All five are the responsibility of RMS.

Table 3: Heritage Listings for McDonald Type Timber Truss Bridges

	SHR	Section 170	LEP
RMS Bridges			
Bridge over Tunks (or Pearces) Creek at Galston Gorge	Yes	Yes	Not listed
McKanes Bridge over the Cox's River near Lithgow	Yes	Yes	Not listed
Junction (or Shelley) Bridge over the Tumut River	Yes	Yes	Tumut
Bridge over Five Day Creek west of Kempsey	Yes	Yes	Kempsey
Crankies Plain Bridge over the Coolumbooka River	Yes	Yes	Bombala

Of the five remaining McDonald trusses, RMS intends to retain three in accordance with the RMS Timber Truss Bridge Conservation Strategy. Five Day Creek Bridge is one which has been demolished since 2013, having been replaced with a concrete bridge many years prior.

Figure 33: McDonald Truss over Five Day Creek with new Concrete Bridge in Background

2.5.1.3 Gradings of Significance in McDonald Type Timber Truss Bridges

Table 4: Gradings of Significance for McDonald trusses with Implications for Conservation

	Primary Characteristic	Implications for Conservation
Exceptional	Principals are significantly longer than diagonals, and are splayed at the base in order to provide lateral stability to the top chord. Timber spacers in the principals are structural and are notched into flitches. Principals have a diagonal timber prop approximately half way along the length and a vertical tension rod also at that location.	This relates to the shape of the truss, and is one way it can be recognised as a McDonald truss. The splayed principals with structural timer spacers are unique and essential to the original design intent, as the lateral stability of the truss depends entirely on these members.
	Designs have two, three or four counterbraced central panels between principals. Double members in central panels have timber spacers at each end, with grain of spacers generally running in direction of the member.	This relates to the shape of the truss, and is one way it can be recognised as a McDonald truss.
	The top chords consist of single large section solid sawn timber members.	Alteration detracts from significance by obscuring the context of this design (plentiful timber). Use of smaller members bolted together to form larger members reduces the strength of the bridge.
	Iron wedges are provided at each panel point for taking up the slackness in the braces	These are unique to the McDonald truss and are essential to the original design intent, and a significant improvement over the Old PWD truss.
High	Bottom chords consist of four lines of bolted sawn timber laminates. Joints for internal laminates occur towards the ends of the span and have no metal splice plates. Joints for external laminates occur toward the centre of the span and there is a large splice plate provided on both sides of the bottom chord to cover both joints.	Original lengths of timber used in the bottom chord are no longer available, but shorter lengths significantly reduce the strength of the bridge, and so some form of strengthening by introduction of other materials is generally necessary.
	Principals bear against long timber butting blocks. Butting blocks are bolted to the bottom chord between timber notches.	With the requirement to introduce other materials into the bottom chord in order to provide for modern vehicular loadings, these connection details will also require modifications. In addition to the introduction of modern materials, the current construction technique of prefabricating either trusses or entire spans will generally mean that bottom chords cannot be continuous over piers, and so the flow of forces may also require change.
	Vertical tension rods at each end of the top chord consist of two rods placed outside the chord and held in place by cast-iron cradles. All other tension rods are single rods installed through holes drilled into the top and bottom chords (similar to the Old PWD).	The original wrought iron tension rods are subject to failure due to deterioration and increased loads, and so these elements are generally replaced with steel tension rods having a greater diameter. The arrangement of tension rods, however, is essential to the original design intent of the McDonald truss.
	No corbels are provided over the piers at the ends of the truss spans.	This is unique to the McDonald truss, and is a visual indicator to the fact that the laminated timber bottom chords are continuous over the piers.
Moderate	Timber cross girders are closely spaced and carry diagonal decking.	These elements contribute to the overall significance of the bridge, but are not essential or unique to this truss type. Other deck systems may give a superior conservation outcome.
	Iron T sections 6” x 4” x ½” are used for sway braces, with sway bracing provided at all internal panel points excluding the two ends of the top chords.	The original sway braces were never intended to provide lateral stability to the truss, as this was provided by the principals. Additional lateral support is required for heavier loads, and so modifications to the sway braces so that they provide lateral restraint are generally required. It would be preferable not to add sway braces at the principals because this distorts the original design intent of principals providing lateral rigidity.

2.6 Timber Truss Bridge Type 3: Allan Truss

Figure 34: Percy Allan

Figure 35: An "Allan" Truss Bridge

Percy Allan, born 12 July 1861 in Sydney, was the youngest son of Maxwell Rennie Allan, later under-secretary, Colonial Secretary's Office, New South Wales. Educated at Calder House, Sydney, Percy Allan joined the Roads Branch, Department of Public Works, as a cadet in 1878.

Percy Allan became Assistant Draftsman in 1882 and Chief Draftsman in 1889. His training by pupillage continued under senior engineers within the department in accordance with the conditions prescribed by the Institution of Civil Engineers, London. Appointed Assistant Engineer for Bridges in 1895, he was promoted a year later to engineer-in-charge of bridge design. In 1900 Allan assumed increased responsibility for rivers, artesian bores, water-supply and drainage. His work included supervising the construction of Sydney's sewerage system with ocean outfalls. In Newcastle from 1908 until 1912 as District then Chief Engineer, among other things, he designed and built additional coal-loading wharves and cranes. From 1917 until he retired in 1927 he was Chief Engineer, National and Local Government Works.

Percy Allan was the first president and a life member of the Northern Engineering Institute and of its successor, the Newcastle division of the Institution of Engineers, Australia. He was also a member of the American Society of Civil Engineers. He had a distinguished career, and left many monuments, having designed nearly six hundred bridges. Dr J.J.C. Bradfield (designer of the Sydney Harbour Bridge) said that whatever success he had attained he owed largely to the assistance he had received from Allan in his youth. Allan was awarded a Telford Premium in 1921 by the Institution of Civil Engineers (London), for a paper submitted on the port improvements at Newcastle. The Telford Premiums were monetary awards for papers considered worthy, and, according to the newspapers of the day, were highly regarded.

In 1893, upon the retrenchment of McDonald, Allan introduced significant changes to the design of both timber beam and timber truss bridges. By this time, he had worked with Bennett for over ten years and with McDonald for almost 15 years. In addition to this, he had worked briefly with Professor Warren, and had had the benefit of de Burgh's assistance for eight years and Dare's for four years. Percy Allan was able to make significant progress in the design of timber truss bridges, as he had access to the best information, including both the structural properties of Australian hardwoods, and 35 years of historical data on various timber designs.

Allan's design was driven by financial constraints. Money was scarce after the bank crashes in London in 1890 and in Australia in 1892-3. Labourers and supervisors were being laid off and engineers (such as McDonald) retrenched or forced into early retirement. Large section timbers were increasingly scarce and therefore expensive. Earlier designs were not intended as permanent structures since the road alignments were still being developed, and so they were expensive to maintain, and often had to be replaced. The focus for Allan in his design was the use of small section timber designed for maintainability. Allan's design philosophy was similar to that of the 19th century architect, landscape gardener and poet, Thomas Pope:

> *"When Time, with hungry teeth, has wrought decay,*
> *Then what will sceptics be dispos'd to say?*
> *Why, "down the Bridge must fall, without repair,*
> *And all the author's pleadings will be air."*
> *Not so, he's better arm'd than you'd expect,*
> *For nought can bring to ruin but neglect;*
> *A means provided, which can never fail,*
> *To keep up strength whate'er the Bridge may ail:*
> *Each log of wood, where'er its station be,*
> *Is safely shifted for a sounder tree…"*

The PWD Report to the NSW Legislative Assembly for the Year 1893-4 details Allan's innovations: "*The standard design for plain beam bridges has, during the past year, been completely recast – corbels having been reduced in length, corbel plates omitted, and keys reduced in number; butt joints in girders have been substituted for the expensive scarfs hitherto in vogue; capwales have been introduced in lieu of capsills, and abutments have been so remodelled as to do away with all sheathing and wing piles the saving in abutments in some cases being as much as 75 per cent., while on an average the alteration in design has effected a saving in the cost of plain beam bridges of some 20 per cent. The type design for truss bridges in use since 1884* [1886] *has been superseded by a truss of more modern design, the principal features of which are: the use of marketable lengths of timber, the adoption of open chords and braces always accessible to the brush, and the ease with which any defective timber can be replaced. In each of the new 90-feet spans there is a saving of 450 cubic feet of timber, while the trusses are capable of carrying 10 feet more roadway than in the old type of truss, thus affording greater travelling facilities at reduced cost. Not only is there a saving in materials in the new type of truss, but a considerable saving is effected owing to the shorter lengths of timber employed and the greater ease in framing together. Altogether the saving effected by the adoption of the new type of truss bridge is on the average about 20 per cent.*"

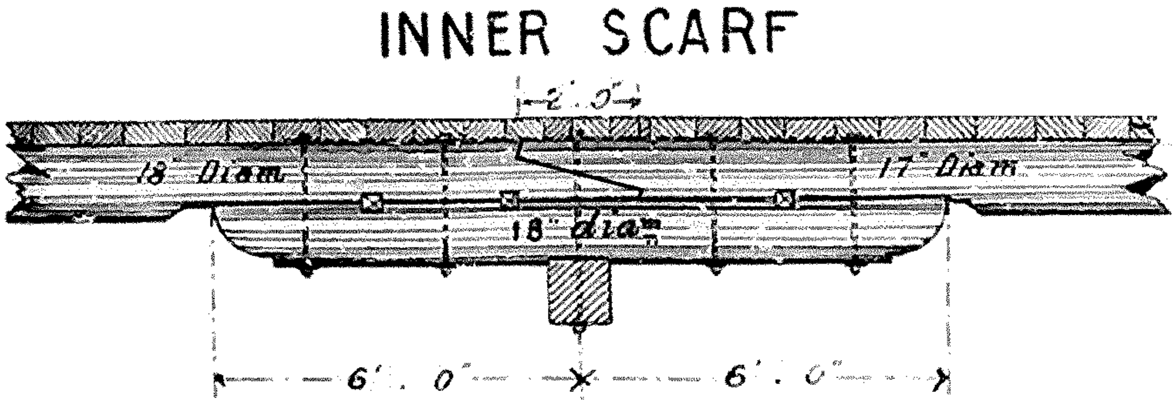


Figure 36: Pre-1893 Plain Beam Bridges: long corbels, numerous shear keys, scarf joints, capsills

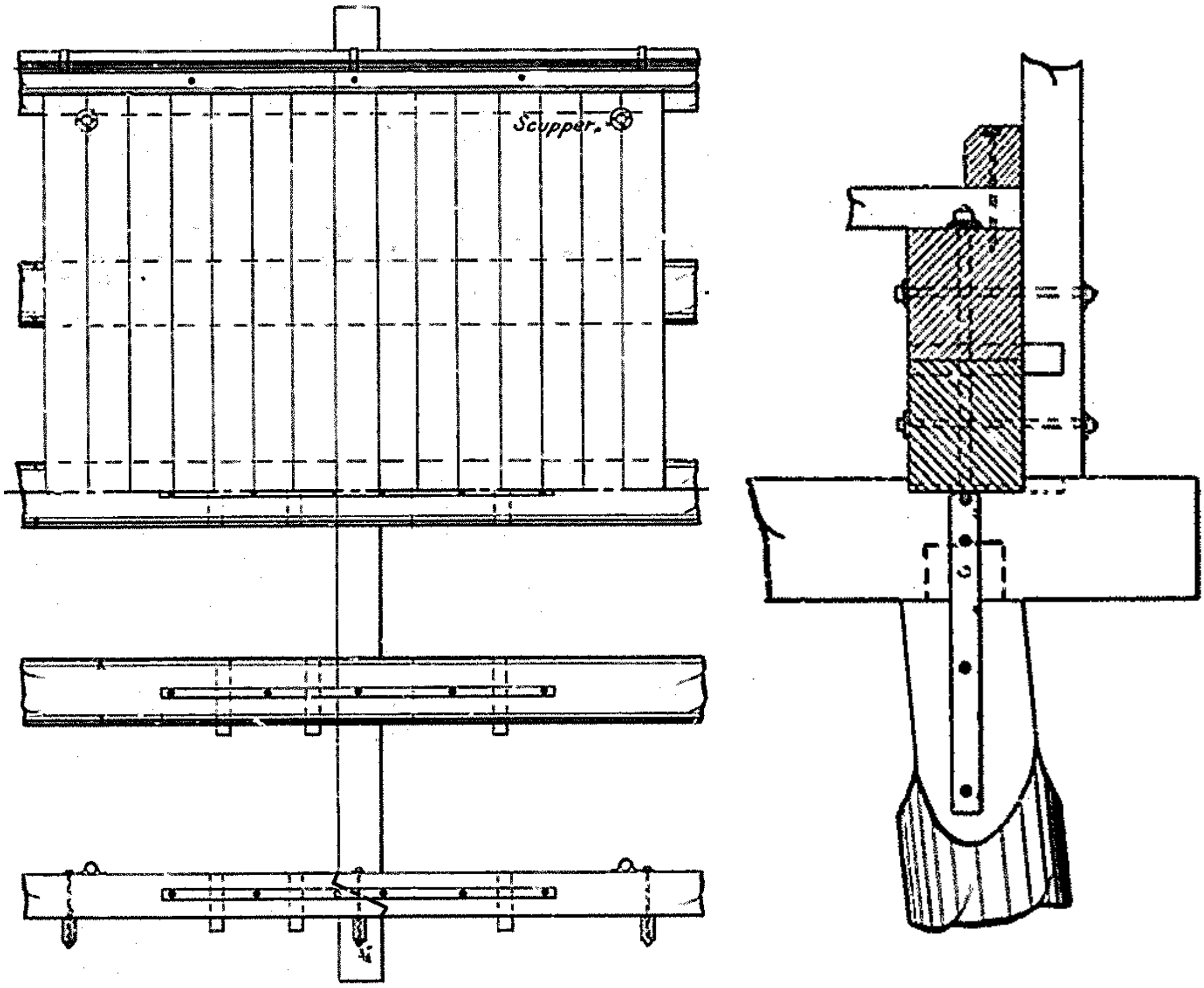


Figure 37: Pre-1893 Plain Beam Bridges: Corbel Plates & Scarf Joints (left) Mortise & Tenon Joint (right)

The bottom chord splices being so critical, Allan tested the full-size joint in a machine specially designed for the purpose. The machine consisted of a heavy ironbark frame and large hydraulic jack. In the three tests conducted, failure occurred by the shearing of the bolts and of the timber between the notches, the recorded results showing an ultimate strength of 151, 160 and 182 tons (153, 162, 185 tonnes) respectively, the size of the timber flitch was 13” x 6” (330 x 150 mm) and the steel plates 12¾” x ½” (324 x 12 mm). Warren noted that for ironbark timber, the shearing resistance along the fibre is generally about 2,000 lbs. per square inch (14 MPa).

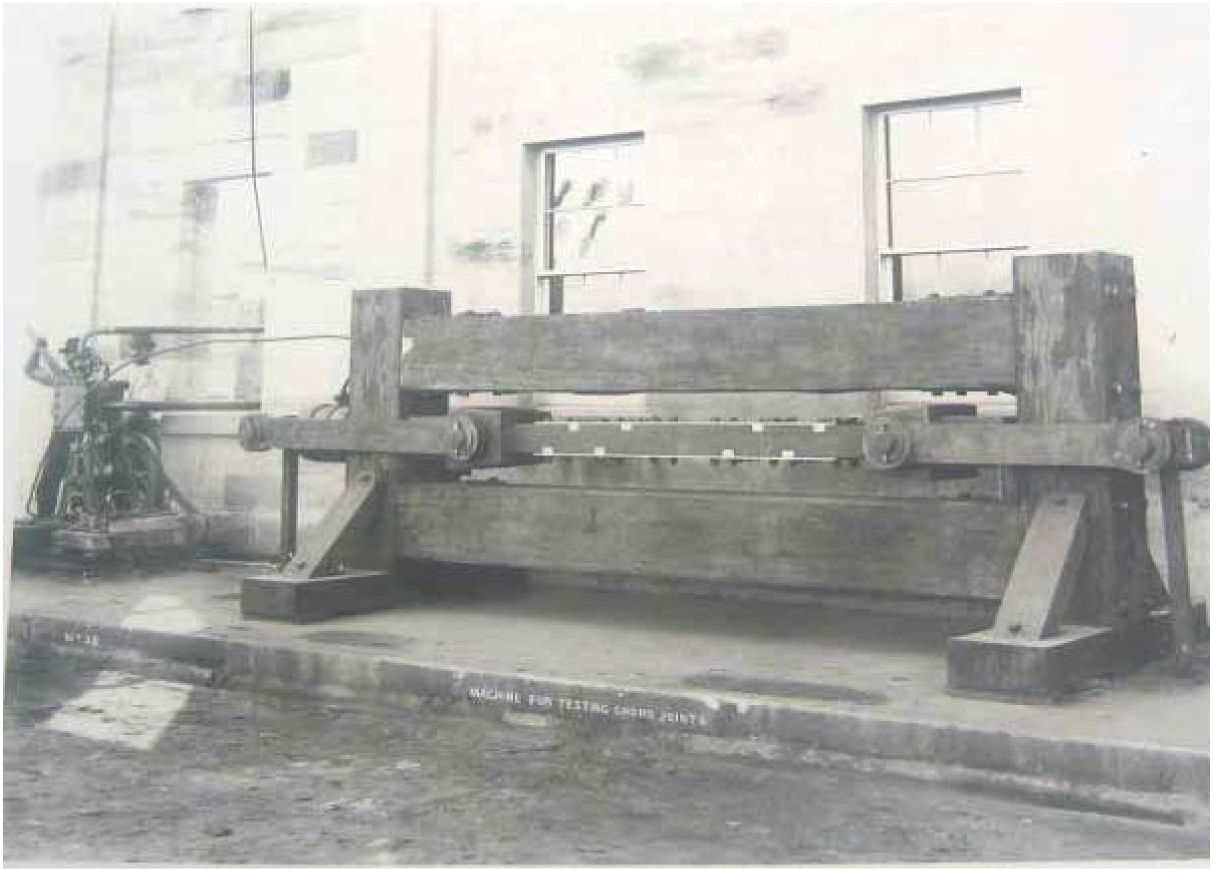


Figure 38: Percy Allan’s Photograph of Chord Testing Machine at Biloela Dockyard (Cockatoo Island)

In addition to designing timber truss bridges, Percy Allan designed the very innovative swing spans for the Pyrmont Bridge and the old Glebe Island Bridge. The fact that Pyrmont Bridge was Australian designed and Australian built was a focal point for national pride at the time of federation. Percy Allan also designed a number of lift spans some of which were even noticed overseas, such as the ones at Swan Hill, Tooleybuc and Dunmore. There are also a number of steel truss bridges which he designed, including Tom Uglys Bridge over the Georges River in southern Sydney and Mulwala Bridge and Abbotsford Bridge over the Murray River.

In his youth Percy Allan was a prominent athlete. A member of the Newtown Football Club, he toured New Zealand in the 1886 Rugby side and later was a referee. He married Alice Mary on 11 November 1890 and had two sons. He and his wife were both keen golfers.

2.6.1 Characteristics of the Allan Type Timber Truss Bridges

The primary characteristics that distinguish Percy Allan's designs for the Allan type timber truss bridges from other timber truss bridges are as follows:

- All diagonals including principals are placed at the same angle.
- Counterbraces are only provided in the middle panel, there are no redundant members.
- The deck consists of transverse decking with two way cross fall. The transverse decking planks rest on longitudinal stringers which span between timber cross girders.
- Timber cross girders are only provided at panel points (no intermediate cross girders).
- All timber members consist of two timber flitches. Principals, diagonals and top chords consist of two flitches bowed around timber spacers to prevent warping and twisting.
- The bottom chord is a tension member not subject to bowing. It consists of two straight timber flitches with a gap between them to allow drainage, maintenance and air flow.
- Instead of using a large number of bolts to transfer the tension force in the bottom chord, as was the theory behind the laminated timber bottom chord, the Allan truss makes use of a special splice plate with shear keys invented and tested by Allan to take the load. Each plate had four shear keys riveted to the plate, which were then let tightly into the timber. Plates were left undrilled until fitted in place, the two plates and timber were drilled through for bolts of a driving fit, which ensured bolts bearing on the plates and the timber.
- Wrought-iron tension rods are located on either side of cross girders, passing through the space between the two flitches of the chords, eliminating bored holes through chords.
- Cast-iron shoes are provided at the top and base of all diagonal members. The horizontal force from the braces is transferred through shear lugs cut into the chords. The use of shoes removes the necessity for timber butting blocks.
- For the longest spans, overhead bracing is used to give lateral stability to the top chord and under-deck wind bracing is provided with metal rods.
- Iron T sections are used for sway braces, with sway bracing sometimes provided at all panel points, and sometimes only at alternating panel points.

2.6.1.1 Conserving Engineering Heritage in Allan Type Timber Truss Bridges

According to the NSW Heritage Division, "the main aim in assessing significance is to produce a succinct statement of significance, which summarises an item's heritage values. The statement is the basis for policies and management structures that will affect the item's future."

Items listed on the RMS Section 170 Heritage and Conservation Register or on the NSW Heritage Division's State Heritage Register will generally have a Statement of Significance which should inform the design process. The Statement of Significance should include not only technical significance, but also historical, associational, aesthetic, social, archaeological, rarity and representativeness where relevant. However, since these structures are engineering heritage, a Statement of Significance for an Allan truss bridge would rightly include the following:

> *As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and with Percy Allan, then Chief Draftsman, an eminent engineer, and the designer of this truss type. Allan trusses were the third in the five-stage development of NSW timber truss road bridges. The trusses took advantage of the high quality NSW hardwoods, known to be among the strongest and most durable in the world. The design is an example of innovative and efficient timber engineering in a time when budgets were tight. The evolution in design shows the growing knowledge of timber as a structural material, the increasing difficulty in obtaining large section long timbers, and the need for durable and maintainable bridge designs.*

The Statement of Significance must inform the design process. Examples of how the Statement of Significance above should inform the conservation of Allan trusses are given here.

The historical context of these bridges is the availability of high quality NSW hardwood and so their conservation should continue the use of NSW hardwood timbers.

These bridges have strong associations with the expansion of the road network and economic activity throughout NSW. Therefore, the conservation of these bridges should retain their use as a vital part of the NSW road infrastructure, which will generally require their strengthening.

These bridges also have strong associations with Percy Allan, and have the opportunity to demonstrate the engineered design details. Although many of the details may have changed throughout the life of the bridge, there is generally sufficient evidence of the original design both in drawings and in old photographs to allow restoration and reconstruction within the bounds of Articles 19 and 20 of the Burra Charter. Therefore, the conservation of these bridges should seek to apply engineering excellence so as not to obscure the work of this eminent engineer.

As timber truss bridges, Allan trusses are aesthetically distinctive and have landmark qualities. However, it is the innovative, economical, durable and maintainable design which is particularly notable. Therefore, the conservation of these bridges should not obscure the original details, particularly those details which display innovations in durability and maintainability of the bridge.

2.6.1.2 Remaining Allan Type Timber Truss Bridges and Listings (as of 2013)

Of over 100 Allan type timber trusses built in New South Wales between 1893 and 1929, 28 remained in 2013. These are listed below and include 19 which are the responsibility of RMS.

Table 5: Heritage Listings for Allan Type Timber Truss Bridges

	SHR	Section 170	LEP
RMS Bridges			
Beryl Bridge over Wyaldra Creek near Gulgong	Not listed	Yes	Not listed
Boonangar Bridge over Barwon River near Boomi	Not listed	Yes	Not listed
Tooleybuc Bridge over the Murray River	Yes	Yes	Not listed
Carrathool Bridge over the Murrumbidgee River	Yes	Yes	Not listed
Bridge over the Abercrombie River near Tuena	Not listed	Yes	Not listed
Victoria Bridge over Stonequarry Creek at Picton	Yes	Yes	Not listed
Wallaby Rocks over the Turon River near Sofala	Yes	Yes	Not listed
Hinton Bridge over the Paterson River	Yes	Yes	Port Stephens
Vacy Bridge over the Paterson River	Yes	Yes	Dungog
Barrington Bridge over the Barrington River	Not listed	Yes	Gloucester
Swan Hill Bridge over the Murray River	Yes	Yes	Not listed
Payten's Bridge, Collett's Crossing, Lachlan River	Not listed	Yes	Not listed
Bridge over Yass River near Gundaroo	Not listed	Yes	Not listed
Thornes Bridge over Mulwaree Ponds near Goulburn	Not listed	Yes	Goulburn Mulwaree
Charleyong Bridge over Mongarlowe River	Not listed	Yes	Not listed
Bridge over the Goodradigbee River at Wee Jasper	Yes	Yes	Not listed
Rossi Bridge over the Wollondilly River near Goulburn	Yes	Yes	Goulburn Mulwaree
Morpeth Bridge over the Hunter River	Yes	Yes	Maitland
Dunmore Bridge over the Paterson River at Woodville	Yes	Yes	Port Stephens
Beryl Bridge over Wyaldra Creek near Gulgong	Not listed	Yes	Not listed
Non-RMS Bridges			
Bridge over Green Gully at Queens Pinch south of Mudgee	Not listed	Not listed	Not listed
Pyrmont Bridge over Darling Harbour	Yes	Yes	Not listed
Duffs Bridge over Dingo Creek near Marlee	Not listed	Not listed	Greater Taree
Marlee Bridge over Dingo Creek at Marlee Flat	Not listed	Not listed	Greater Taree
Bridge over Styx River near Jeogla	Not listed	Not listed	Not listed
Bridge over Molonglo River near Foxlow	Not listed	Not listed	Not listed
Bridge over Mataganah Creek near Wyndham	Not listed	Not listed	Bega Valley
Tharwa Bridge over the Murrumbidgee River in the ACT	N/A	N/A	N/A
Hampden Bridge, Murrumbidgee River, Wagga Wagga	Not listed	Not listed	Wagga Wagga

2.6.1.3 Gradings of Significance in Allan Type Timber Truss Bridges

Table 6 provides generic gradings of significance for the primary characteristics of an Allan truss.

Table 6: Gradings of Significance for Allan trusses with Implications for Conservation

	Primary Characteristic	Implications for Conservation
Exceptional	All diagonals including principals are placed at the same angle. For the longest spans, overhead bracing is used to give lateral stability to the top chord.	This relates to the shape of the truss, and is one way it can be recognised as an Allan truss.
	Counterbraces (timber diagonals) are only provided in the middle panel, there are no redundant members.	This relates to the shape of the truss, and is one way it can be recognised as an Allan truss. It is essential to the original design intent, which was to maximise economy by minimising materials.
	All timber members consist of two timber flitches. Principals, diagonals and top chords consist of two flitches bowed around timber spacers to prevent warping and twisting.	This relates to the shape of the truss, and is one way it can be recognised as an Allan truss. The use of paired timbers for all members is essential to the original design intent, relating to designing for durability and maintainability.
	Instead of using a large number of bolts to transfer the tension force in the bottom chord, as was the theory behind the laminated timber bottom chord, the Allan truss makes use of a special splice plate with shear keys invented and tested by Allan to take the load.	This detail was first invented by Allan for use in the Allan truss, but was afterwards used by the rail authorities as well as overseas. The splice plates demonstrate technical excellence, innovation and achievement, and therefore contribute directly to the cultural significance of the Allan truss. Given, however, their tendency to fail under modern loads, some modifications to the load paths around this detail will generally be necessary.
High	Timber cross girders are only provided at panel points (no intermediate cross girders).	Limiting cross girders to panel point locations is essential to the original design intent. However, due increased loads, timber cross girders are generally under-capacity and it is appropriate to replace them with steel cross girders of hollow rectangular section painted the same colour as the timber to minimise the visual impact on the truss.
	Wrought-iron tension rods are located on either side of cross girders, passing through the space between the two flitches of the chords, eliminating bored holes through chords.	The locations and load paths of the tension rods are essential to the original design intent. However, the wrought iron is generally under-capacity and subject to sudden brittle failure, and it is appropriate to replace them with steel rods.
	The bottom chord is a tension member not subject to bowing. It consists of two straight timber flitches with a gap between them to allow drainage, maintenance and air flow.	The timber bottom chord and its configuration (two parallel separated timber flitches without spacers) are essential to the original design intent. However, due to their susceptibility to sudden and brittle failure, some form of strengthening by introduction of other materials is necessary.
	Cast-iron shoes are provided at the top and base of all diagonal members. The horizontal force from the braces is transferred through lugs 1½" (38mm) deep into the chords, and where two lugs are necessary, the deeper lug is at the back of the casting so as to distribute the force over a larger area and reduce the risk of failure by shearing between the lugs. The use of shoes removes the necessity for timber butting blocks.	The use of metal shoes to transfer the loads is essential to the original design intent, as is the shape of the shoes as they were designed. However, cast iron is a brittle material and is susceptible to sudden and brittle fracture, causing loss of stability to the bridge. It is appropriate to replace them with either a modern ductile cast iron or with welded steel replicas, painted black.
Moderate	The deck consists of transverse rather than diagonal decking. The transverse decking planks rest on longitudinal stringers which span between timber cross girders.	These elements contribute to the overall significance of the bridge, but are not essential or unique to this truss type. Other deck systems may give a superior conservation outcome.
	Iron T sections are used for sway braces, with sway bracing sometimes provided at all panel points, and sometimes only at alternating panel points.	The original sway braces were never intended to provide lateral stability to the truss. Additional lateral support is required for heavier loads, and so modifications to the sway braces so that they provide lateral restraint are generally required.

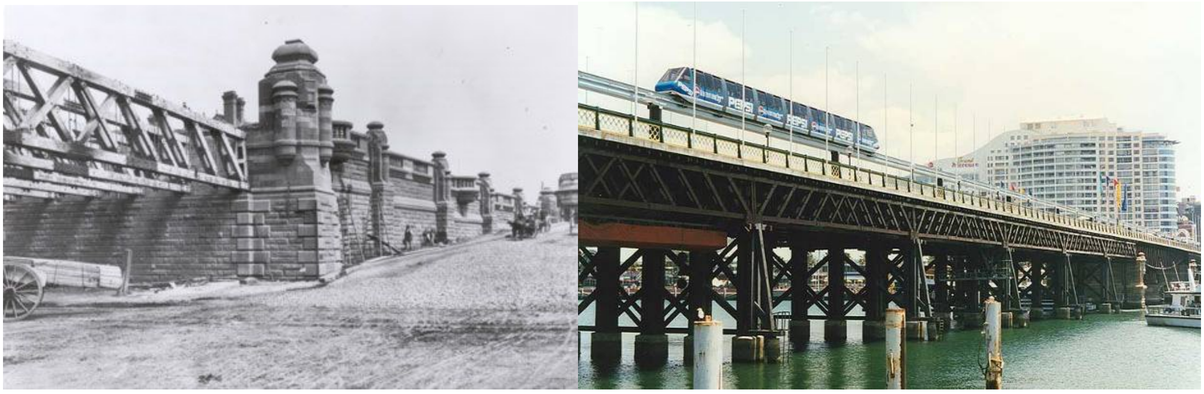
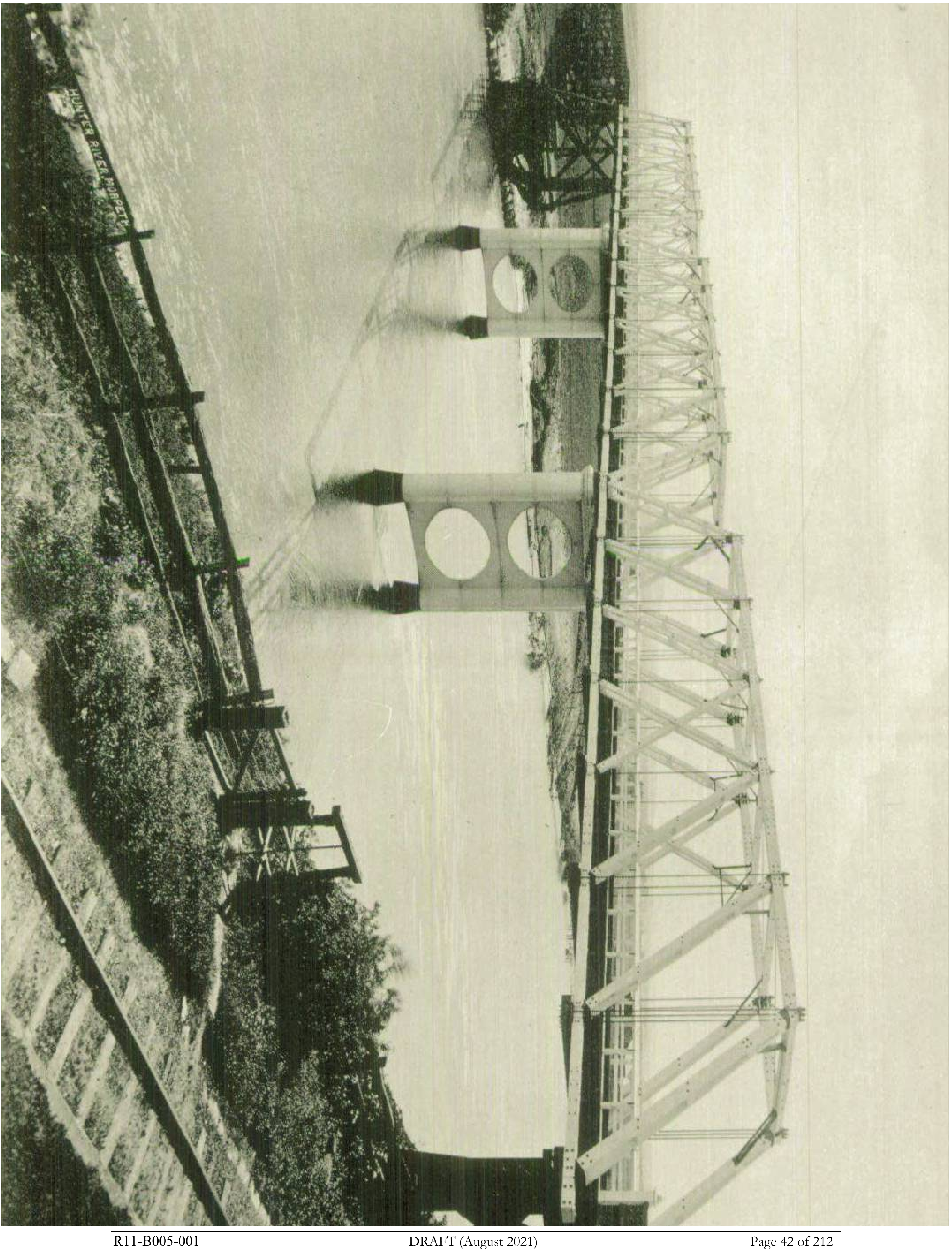
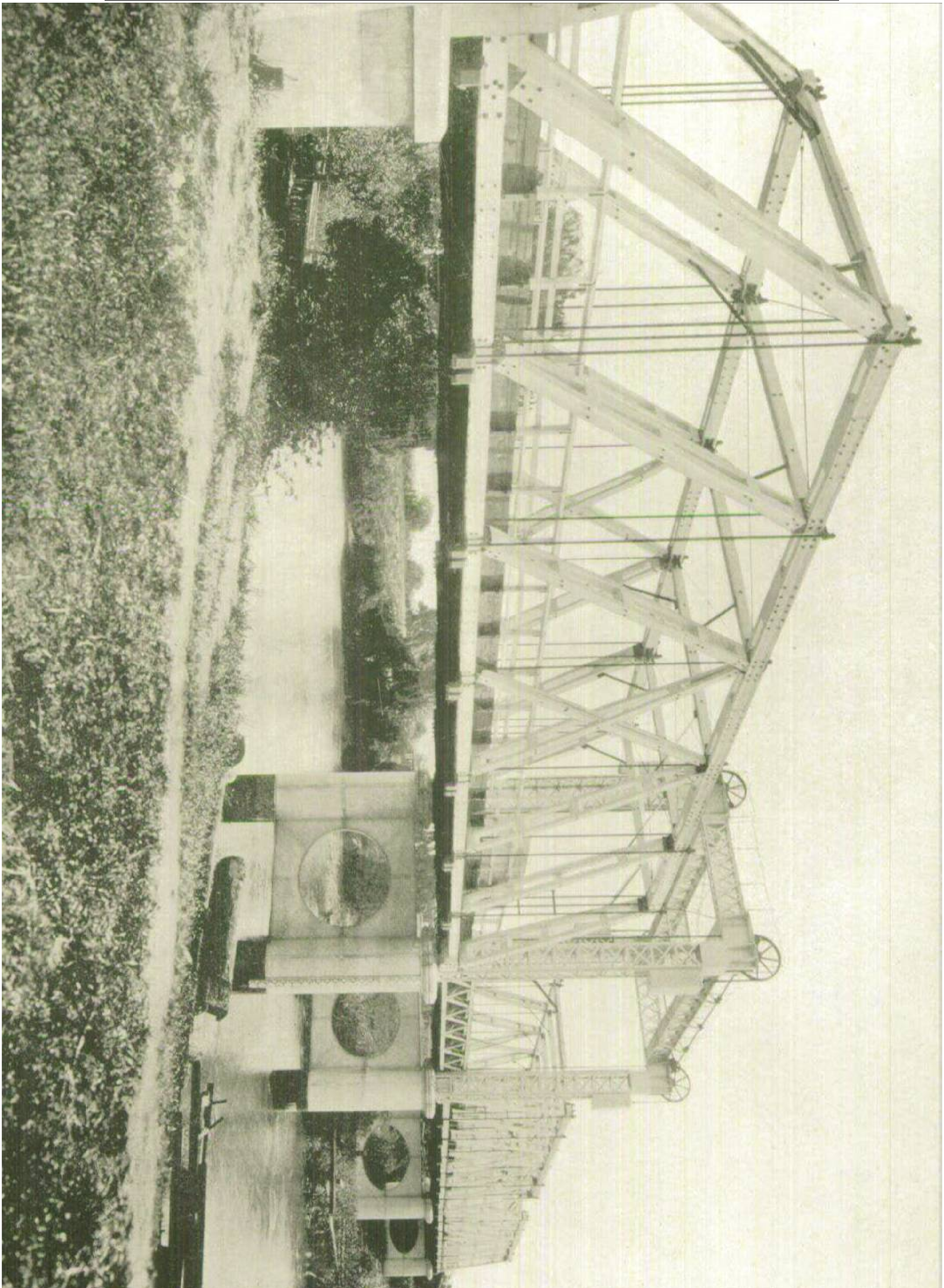


Figure 39: Allan Truss: Pyrmont Bridge with Six Allan Trusses Below the Deck on Each Span

Figure 40: Allan Truss: Vacy Bridge over the Patterson River near Dungog

Figure 41: Allan Truss: Morpeth Bridge over the Hunter River near Maitland with Overhead Bracing


HUNTER RIVER, MORPETH



2.7 Timber Truss Bridge Type 4: de Burgh Truss

Figure 42: E. M. de Burgh

Figure 43: A "de Burgh" Truss Bridge

Ernest Macartney de Burgh was born at Sandymount County Dublin, Ireland on 18 January 1863. He was educated at Rathmines School and the Royal College of Science, Ireland. After graduating, he was engaged for a time on railway work in Ireland, and later came to New South Wales, joining the Public Works Department on survey and construction work in 1885. Within two years he was in charge of the construction of steel bridges across the Murrumbidgee and Snowy Rivers, and then designed and superintended the construction of many other bridges throughout the State. However, de Burgh is probably best remembered for his contribution to water supply and conservation, having designed and constructed many great engineering works.

That de Burgh made his mark in the country of his adoption is no wonder: his ability and his character rendered mediocrity impossible. A typical Irishman in many respects, de Burgh was a deservedly popular officer. In his dealings with the many men who come under him he had a way which won the goodwill of all; and which enabled him to quickly arrange any of those little difficulties which are apt to crop up where large numbers of men are engaged. From his officers he got the best work by creating a feeling of companionship in all things. It was his breezy personality and ready wit that made an impression upon most people with whom he came in contact. Contractors knew that if they treated him fairly, he would give them also a fair deal; but woe betide the contractor or the officer either, who failed to please him. The edge of his tongue could then be very rough, however charming his manner when things were going satisfactorily. In the engineering profession his name ranked high, not only in Australia, but also in Great Britain, where he was twice awarded Telford Premiums for papers contributed to the Institution of Civil Engineers, London. A man of large ideas and wide vision, it was the initiation and carrying out of his big schemes that "The Chief" as he was called, found his greatest delight, and the great public works which he constructed stand as fitting monuments to his skill.

Henry Harvey Dare had significant involvement in the design of the de Burgh type timber truss bridges, but he designed them acting under de Burgh with his assistance and advice, and Percy Allan attributed the introduction of the new design to de Burgh, so it is not inappropriate to call them de Burgh trusses. These bridges are a composite truss design based on the American Pratt type truss, in which the bottom chords and diagonals are of steel and the verticals and top chords of timber. The connection of diagonal tension rods with the bottom chord is effected with pins.

By the very end of the 1800s, it had been found that, despite Allan's attention to detail and significant innovations introduced in his Allan truss, in almost every case the timber bottom chord had been the first member of the truss to fail, and the flitches, being in tension, were difficult to replace. Another reason for the introduction of the composite truss, according to Dare, was the extensive timber export trade, which had made it increasingly expensive and difficult to obtain lower chord timbers, which had to be of the best quality ironbark.

In addition to the introduction of a new truss type, de Burgh also brought innovations to the substructure design of timber truss bridges with his use of Monier Pipes as both a pile covering, and in place of cast-iron for cylinder foundations. In 1899 de Burgh reported to the Legislative Assembly as follows: "*Cockle Creek Bridge and the Monier system – This bridge, though not of importance in point of magnitude… is of considerable interest, owing to the use in its construction of Monier cylinders in lieu of cast-iron… The difficulty of protecting timber piers in saltwater from the attacks of the cobra leads to the use of the cylinders, generally of cast-iron filled with concrete, for important piers such as those under truss spans, and the cost of the cast-iron cylinders has been a very serious item. The success of the Monier system in connection with pipes of all sizes suggested that it might be used as a substitute of cast-iron in cylinders, and it was tried for the first time at Cockle Creek as now described. The main piers of the bridge each consist of two cylinders 3 ft. 6 in. internal diameter, and 2¼ inch thick constructed on the Monier principle, and having one layer of wire-netting (1 inch mesh and 16 gauge), and two spirals of 10 gauge steel wire wound completely round the cylinder, the turns being 1 inch apart. The longitudinal connection is formed by six steel bars 1¼ inch by ¼ inch placed between the wire spirals; these bars are so arranged that those of adjoining lengths of cylinder can be coupled together by means of a small fish-plate and steel wedges… The cost of these cylinders delivered at site was 24s. per foot, as against £3 per foot for cast-iron cylinders of the usual type, making a saving of £264 5s. on these two small piers alone, and there is no doubt that their use in suitable localities will result in a very large saving in future…*"

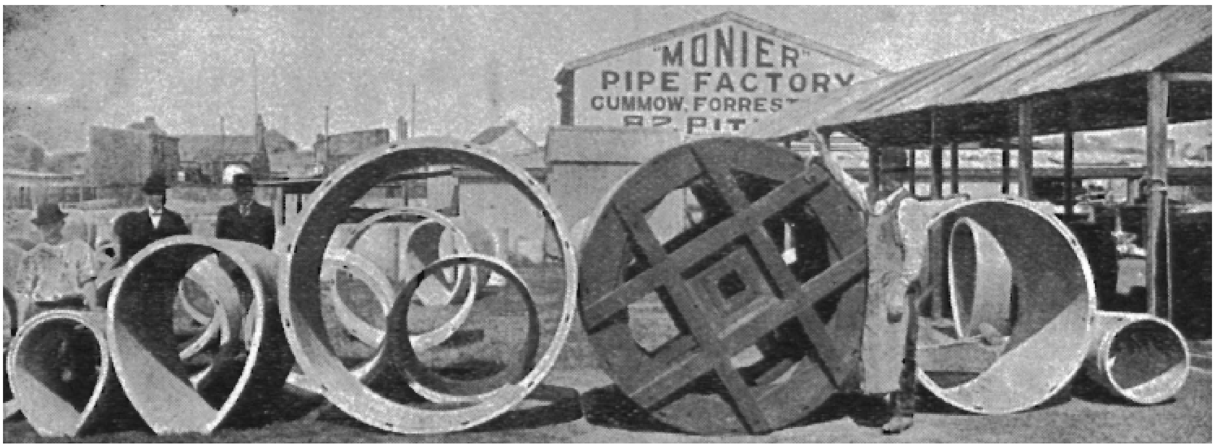


Figure 44: Monier Pipes and Cylinders at Gummow, Forrest & Co Monier Pipe Factory

2.7.1 Characteristics of the de Burgh Type Timber Truss Bridges

The primary characteristics that distinguish Ernest Macartney de Burgh's designs for the de Burgh type timber truss bridges from other timber truss bridges are as follows:

- Timber struts are vertical, while tension rods are diagonal. Timber struts consist of shorter lengths of timber due to the fact that they are vertical rather than diagonal, and this arrangement, along with under-deck bracing, gives additional stiffness over Allan's design.
- Bottom chords and tension rods are generally both steel, with pinned connections.
- Trusses incorporate the improved features of Percy Allan's designs such paired timber flitches curved to prevent bowing and cross girders at panel points.
- Load transfer between timber members is through cellular cast-iron anchor blocks.
- For wider bridges (e.g. Barham) de Burgh provided steel cross girders rather than timber.
- Top chords consist of two parallel timber flitches with channel shaped splice plates.
- Gaps between flitches of the top chord are maintained by castings and bolts.
- While Allan detailed transverse timber decking on stringers, de Burgh reintroduced a small angle on the transverse decking so that it was not perpendicular to the stringers.
- Metal T sections are used for sway braces located at alternating panel points.

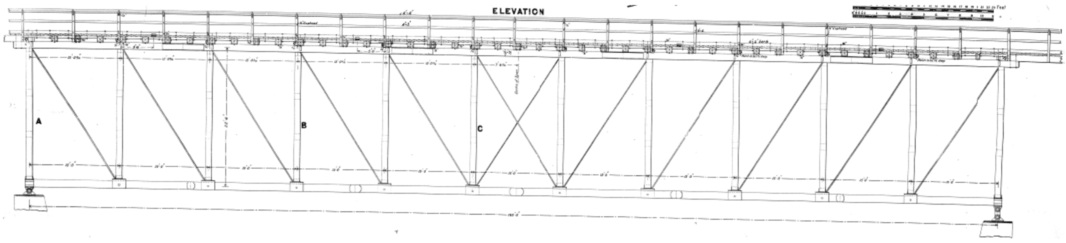


Figure 45: de Burgh Truss: de Burgh's Bridge 165 ft span over the Lane Cove River (truss under deck)

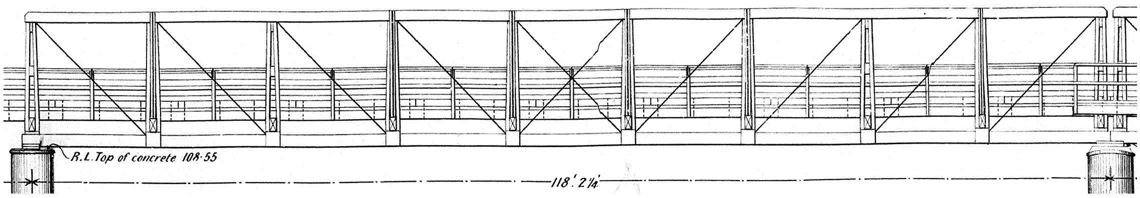


Figure 46: de Burgh Truss: Bridge over the Macdonald River at St Albans (verticals of four flitches)

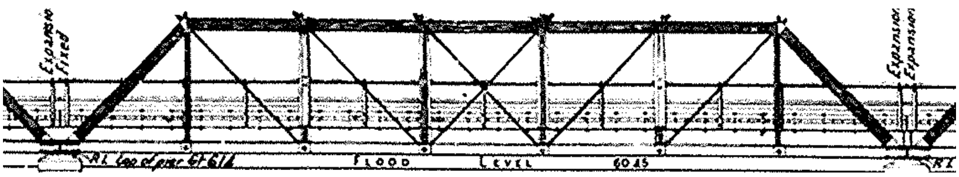


Figure 47: de Burgh Truss: Bridge over Queanbeyan River at Queanbeyan (diagonal principals)

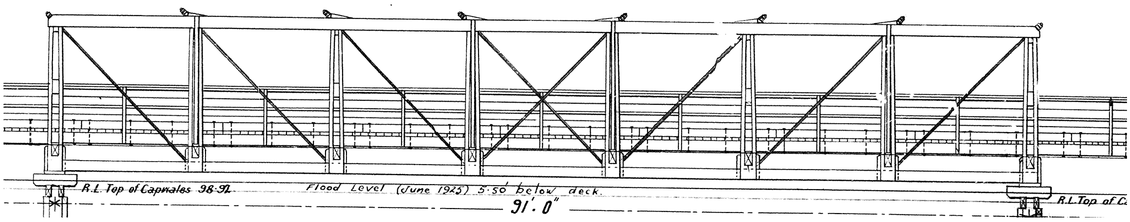


Figure 48: de Burgh Truss: Lansdowne Bridge over Mulwaree Ponds near Goulburn (typical)

2.7.1.1 Conserving Engineering Heritage in de Burgh Type Timber Truss Bridges

According to the NSW Heritage Division, "the main aim in assessing significance is to produce a succinct statement of significance, which summarises an item's heritage values. The statement is the basis for policies and management structures that will affect the item's future."

Items listed on the RMS Section 170 Heritage and Conservation Register or on the NSW Heritage Division's State Heritage Register will generally have a Statement of Significance which should inform the design process. The Statement of Significance should include not only technical significance, but also historical, associational, aesthetic, social, archaeological, rarity and representativeness where relevant. However, since these structures are engineering heritage, a Statement of Significance for a de Burgh truss bridge would rightly include the following:

> *As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and with Ernest Macartney de Burgh, then Assistant Engineer for Bridges, one of the ablest engineers in Australia, and the designer of this truss type. De Burgh trusses were the fourth in the five-stage development of NSW timber truss road bridges. The trusses took advantage of the high quality NSW hardwoods and also steel, which had become increasingly economical. The design is an example of engineering excellence, using a wide range of materials each to their best effect. The evolution in design shows the increasing difficulty in obtaining quality hardwood timbers, as well as problems with the previous timber bottom chords.*

The Statement of Significance must inform the design process. Examples of how the Statement of Significance above should inform the conservation of de Burgh trusses are given here.

The historical context of these bridges is the availability of high quality NSW hardwood and so their conservation should continue the use of NSW hardwood timbers.

These bridges have strong associations with the expansion of the road network and economic activity throughout NSW. Therefore, the conservation of these bridges should retain their use as a vital part of the NSW road infrastructure, which will generally require their strengthening.

These bridges also have strong associations with Ernest Macartney de Burgh, and have the opportunity to demonstrate the engineered design details. Although many of the details may have changed throughout the life of the bridge, there is generally sufficient evidence of the original design both in drawings and in old photographs to allow restoration and reconstruction within the bounds of Articles 19 and 20 of the Burra Charter. Therefore, the conservation of these bridges should seek to apply engineering excellence so as not to obscure the work of one of Australia's ablest engineers.

As timber truss bridges, de Burgh trusses are aesthetically distinctive and have landmark qualities. However, it is the engineering excellence which is particularly notable. Therefore, the conservation of these bridges should not obscure the original details and use of materials.

2.7.1.2 Remaining de Burgh Type Timber Truss Bridges and Listings (as of 2013)

Of approximately 20 de Burgh type timber trusses built in New South Wales between 1900 and 1905, ten remained in 2013. These are listed below and all but one are the responsibility of RMS.

Table 7: Heritage Listings for de Burgh Type Timber Truss Bridges

	SHR	Section 170	LEP
RMS Bridges			
Beckers Bridge over Webbers Creek near Singleton	Yes	Yes	Not listed
Bridge over the Crookwell River at James Park	Not listed	Yes	Upper Lachlan
Lansdowne Bridge over Mulwaree Ponds at Goulburn	Not listed	Yes	Goulburn Mulwaree
Bridge over Glennies Creek at Middle Falbrook	Yes	Yes	Not listed
Tabulam Bridge over the Clarence River	Yes	Yes	Not listed
Cobram Bridge over the Murray River	Not listed	Yes	Not listed
Barham Bridge over the Murray River	Yes	Yes	Not listed
Holman Bridge over the Lachlan River at Gooloogong	Not listed	Yes	Not listed
St Albans Bridge over the Macdonald River	Yes	Yes	Not listed
Non-RMS Bridges			
Gillies Bridge over Black Creek, Rothbury	Not listed	Not listed	Cessnock City

Figure 49: Gillies Bridge - Short Span Council de Burgh truss over Black Creek near Cessnock

2.7.1.3 Gradings of Significance in de Burgh Type Timber Truss Bridges

Table 8: Gradings of Significance for de Burgh trusses with Implications for Conservation

	Primary Characteristic	Implications for Conservation
Exceptional	Timber struts are vertical, while tension rods are diagonal. Timber struts consist of shorter lengths of timber due to the fact that they are vertical rather than diagonal, and this arrangement, along with under-deck bracing, gives additional stiffness over Allan's design.	This relates to the shape of the truss, and is one way it can be recognised as a de Burgh truss.
	Trusses incorporate the improved features of Percy Allan's designs such paired timber flitches curved to prevent bowing and cross girders at panel points.	This is essential to an understanding of the development of timber truss bridge design in New South Wales, where each designer built off the detailing and experience of the previous designs.
	Top chords consist of two parallel timber flitches with channel shaped splice plates. Gaps between flitches of the top chord are maintained by castings and bolts.	This relates to the shape of the truss, and is one way it can be recognised as a de Burgh truss. The use of metal rather than timber for the distance pieces in the top chord is reflective of de Burgh's general move from timber to other materials due to increasing difficulties in sourcing quality timber.
High	Load transfer between members is through cellular cast-iron anchor blocks.	The use of metal anchor blocks to transfer the loads is unique to this truss type and essential to the original design intent, as is the shape of the shoes as they were designed. However, cast iron is a brittle material and is susceptible to sudden and brittle fracture. It is appropriate to replace them with modern ductile cast iron replicas.
	Bottom chords and tension rods are generally both steel, with pinned connections.	Although some of the early designs for de Burgh trusses made use of wrought iron rather than steel, the remaining de Burgh trusses have steel tension rods and bottom chords. Due to increased loads these members are often found to be under-capacity and may require replacement with larger section steel members. Due to the pinned connection details in the de Burgh truss, the tension rods tend to vibrate excessively, sometimes banging against other metal components, causing loss of section over time. Some of the detailing may require improvement in order to prevent this happening in the future.
	For wider bridges (e.g. Barham) de Burgh provided steel cross girders rather than timber.	This relates to the difficulties in obtaining large section long timbers for the cross girders of wider bridges, so that steel was more economical. Due to increasing loads, these are generally found to be under-capacity and it is appropriate to replace them with stronger steel cross girders designed to reflect the original shape and intent of the original.
Moderate	While Allan detailed transverse timber decking on stringers, de Burgh reintroduced a small angle on the transverse decking so that it was not perpendicular to the stringers.	These elements contribute to the overall significance of the bridge, but are not essential or unique to this truss type. Other deck systems may give a superior conservation outcome.
	Metal T sections are used for sway braces located at alternating panel points	The original sway braces were never intended to provide lateral stability to the truss. Additional lateral support is required for heavier loads, and so modifications to the sway braces so that they provide lateral restraint are generally required. Often not only the details but also the number and locations of sway braces need modifications.

2.8 Timber Truss Bridge Type 5: Dare Truss

Figure 50: Henry Harvey Dare

Figure 51: A "Dare" Truss Bridge

Henry Harvey Dare was born at Goulburn, New South Wales, on the 25th August 1867. He was educated at Sydney Grammar School. He graduated from the University of Sydney in 1888 and took a master's degree from the same institution in 1894, winning a University Medal on each occasion. In 1888 he was appointed an assistant astronomical observer at the Sydney Observatory, and in the following year entered the Public Works Department as a draughtsman, where he was engaged in the design of bridges until 1904, after which he was placed in charge of the design of irrigation and drainage works, and was later Senior Commissioner of the Water Conservation and Irrigation Commission. After retiring from the PWD in 1934, he acted as Consulting Engineer in connection with numerous important works throughout Australia.

Dare was prominent in professional bodies, giving numerous technical papers, he was the New South Wales representative in Australia on the Home Council of the Institution of Civil Engineers, London, and was a member of the first council of the Institution of Engineers, Australia. He was awarded the Telford Premium twice, as well as the Peter Nicol Russell medal (which is the Institution of Engineers, Australia's most prestigious award) in 1930. He also has a town, Dareton on the Murray River in far west New South Wales, named after him.

In 1903, at 36 years old, Dare was in charge of highway bridge design, and took the opportunity to change the composite truss, returning to an arrangement similar to the Allan Truss, with diagonal compression members and vertical tension members, but substituting a pair of steel channels for the timber bottom chord, and redesigning the connections to eliminate the pins of the de Burgh Truss. Dare did away with the bowed flitches, simplifying the geometry by designing only straight timbers, and he also experimented with different kinds of shoes on some of his trusses, although the majority of his trusses have shoes similar to those in Allan trusses.

2.8.1 Characteristics of the Dare Type Timber Truss Bridges

The primary characteristics that distinguish Henry Harvey Dare's designs for the Dare type timber truss bridges from other timber truss bridges are as follows:

- Trusses look from a distance to be very similar to an Allan truss, and incorporate the improved features of Allan's and de Burgh's designs such as spaced timber top chords and diagonals, cast-iron shoes, sway braces, wind bracing, and cross girders at panel points.
- The primary differences between the Dare and the Allan truss is that Dare truss bottom chords consist of steel channels, and all timber flitches are straight instead of bowed.
- Bottom chords and tension rods are both steel, but without de Burgh pinned connections.
- Top chords consist of two parallel timber flitches with channel shaped splice plates.
- While Allan detailed transverse timber decking on stringers, Dare usually followed de Burgh's small angle on the transverse decking so that it was not perpendicular to the stringers.

Figure 52: Opening of one of the first Dare Trusses: Bridge over the Macdonald River at Bendemeer

Figure 53: Bridge over the Macdonald River at Bendemeer 100 years after construction

2.8.1.1 Conserving Engineering Heritage in Dare Type Timber Truss Bridges

A Statement of Significance for a Dare truss bridge would rightly include the following:

> *As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and with Henry Harvey Dare, then Assistant Engineer, a prominent engineer with a distinguished career, and the designer of this truss type. Dare trusses were the last in the five-stage development of NSW timber truss road bridges. The trusses took advantage of the high quality NSW hardwoods and also of steel which had become increasingly economical. The design is an example of engineering excellence, using a wide range of materials each to their best effect. The evolution in design shows a combination of the best aspects from the de Burgh and Allan trusses, while avoiding the primary problems with each of them.*

The Statement of Significance must inform the design process. Similar to the other truss types, the conservation of these bridges should continue the use of NSW hardwood timbers, and retain their use as a vital part of the NSW road infrastructure. The conservation of these bridges should seek to apply engineering excellence so as not to obscure the work of this distinguished engineer.

2.8.1.2 Gradings of Significance in Dare Type Timber Truss Bridges

Table 9: Gradings of Significance for Dare trusses with Implications for Conservation

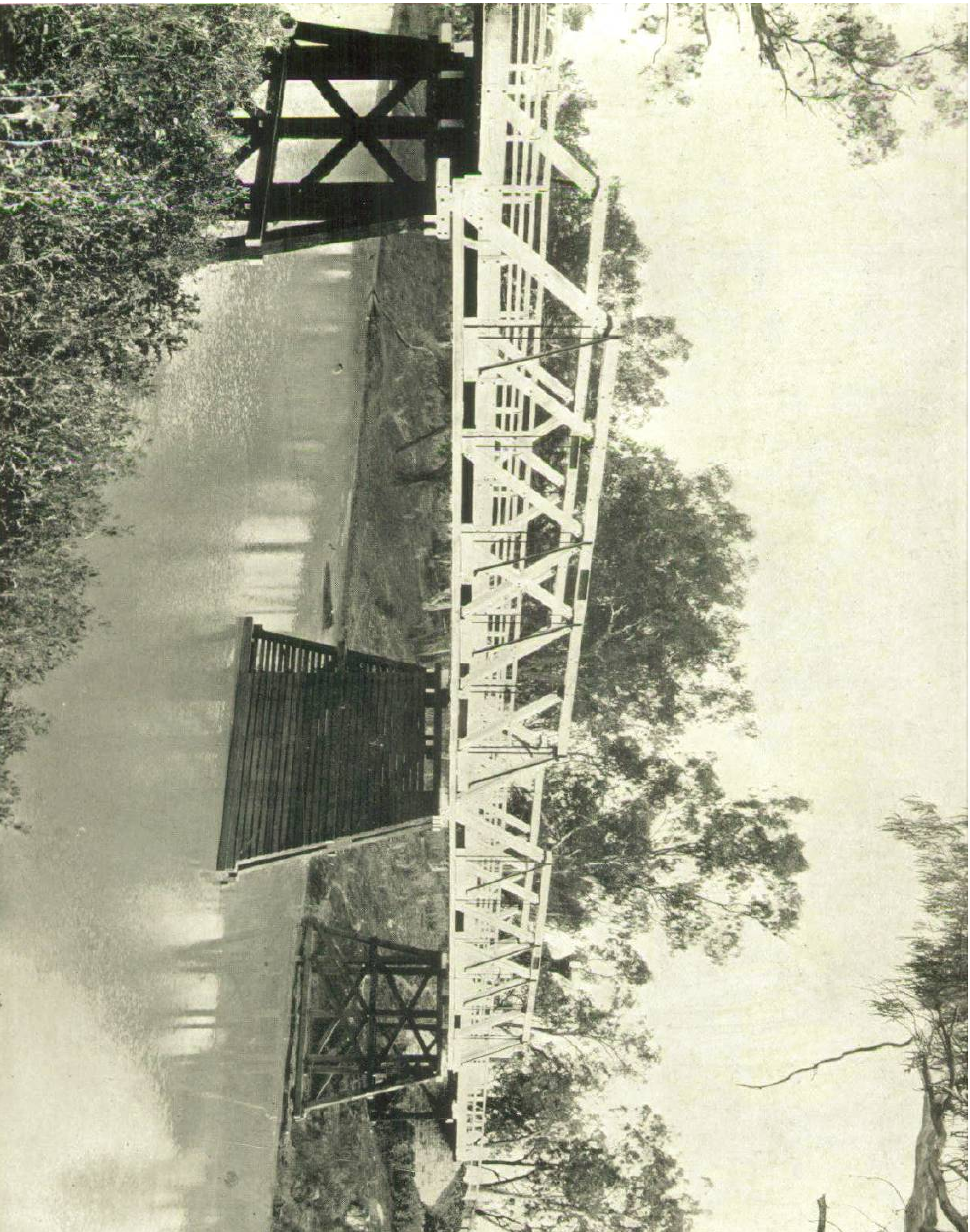
	Primary Characteristic	Implications for Conservation
Exceptional	The primary differences between the Dare and the Allan truss is that Dare truss bottom chords consist of steel channels, and all timber flitches are straight instead of bowed. Top chords consist of two parallel timber flitches with channel shaped splice plates.	This relates to the shape of the truss, and is one way it can be recognised as a Dare truss. This is essential to an understanding of the development of timber truss bridge design in New South Wales, where each designer built off the detailing and experience of the previous designs. De Burgh had introduced straight timbers for the top chord rather than Allan's bowed timbers, and since this had no ill effect, Dare extended the concept beyond the top chords to the principals and diagonals. Dare used similar splice details as de Burgh, but returned to timber spacers in the top chord rather than de Burgh's cast iron distance pieces. The steel sections in the top chord splices and the bottom chords often display foundry marks, which are critical for understanding the original source, and should be retained if possible. Where original steel fabric displaying foundry marks must be replaced due to deterioration or lack of capacity, the foundry marks should be recorded and the information made readily available.
High	Trusses look from a distance to be very similar to an Allan truss, and incorporate the improved features of Allan's and de Burgh's designs such as spaced timber top chords and diagonals, cast-iron shoes, sway braces, wind bracing, and cross girders at panel points.	This is essential to an understanding of the development of timber truss bridge design in New South Wales, where each designer built off the detailing and experience of the previous designs. The same implications for conservation apply to these details as they did for the Allan truss, including requirements to modify sway braces, upgrade cross girders and change the materials of the shoes in order to carry current vehicular loads.
	Bottom chords and tension rods are both steel, but without de Burgh pinned connections.	Due to increased loads these members are often found to be under-capacity and may require replacement with larger section steel members.
Mod.	Allan detailed transverse timber decking on stringers but Dare usually followed de Burgh's small angle on the transverse decking, not perpendicular to the stringers.	These elements contribute to the overall significance of the bridge, but are not essential or unique to this truss type. Other deck systems may give a superior conservation outcome.

2.8.1.3 Remaining Dare Type Timber Truss Bridges and Listings (as of 2013)

Of approximately 40 Dare type timber trusses built in New South Wales between 1905 and 1936, 19 remained in 2013. These are listed below and include 13 which are the responsibility of RMS.

Table 10: Heritage Listings for Old PWD Type Timber Truss Bridges

	SHR	Section 170	LEP
RMS Bridges			
Warroo Bridge over Lachlan River	Not listed	Yes	Not listed
Cooreei Bridge over Williams River at Dungog	Yes	Yes	Not listed
Korns Crossing over Rous River at Crystal Creek	Not listed	Yes	Not listed
Briner Bridge over Upper Coldstream River	Not listed	Yes	Not listed
Border Bridge over Barwon River at Mungindi	Not listed	Yes	Not listed
Coonamit Bridge over Wakool River	Yes	Yes	Not listed
Rawsonville Bridge over Macquarie River	Not listed	Yes	Not listed
Gee Gee Bridge over Wakool River	Yes	Yes	Not listed
Scabbing Flat Bridge over Macquarie River	Not listed	Yes	Wellington
New Buildings Bridge over Towamba River	Yes	Yes	Not listed
Bulga Bridge over Wollombi Brook	Yes	Yes	Not listed
Colemans Bridge over Leycester Creek at Lismore	Yes	Yes	Lismore
Bridge over Sportsmans Creek at Lawrence	Not listed	Yes	Clarence Valley
Non-RMS Bridges			
Junction Bridge over Rouchel Brook	Not listed	Not listed	Not listed
Birrie Bridge over Birrie River at Goodooga	Not listed	Not listed	Not listed
Bells Bridge over Hunter River at Moonan Flat (remains)	Not listed	Not listed	Not listed
Woolbrook Bridge over Macdonald River	Not listed	Not listed	Not listed
Cameron Bridge over Rouchel Brook	Not listed	Not listed	Not listed
Bendemeer Bridge over Macdonald River	Not listed	Not listed	Tamworth



2.9 Timber Girder Bridges and Approach Spans

The construction details prior to 1893 consisted of a series of timber trestles each with three to five driven timber piles. The piles were capped by a headstock which was a single piece of timber sitting over the tops of the piles and attached by internal mortise and tenons and external strap bolts. The piles were then braced on the outsides by opposite inclined timber cross bracing. For a low-level bridge, the cross bracing was sometimes omitted whereas for a high-level bridge there could be two or more levels of cross-bracing separated by a pair of horizontal wales, one on each side of the piles, and with another pair just above ground level. Supported on the headstock, and at right angles to it, would be a set of corbels. Their number and location would be the same as the main longitudinal beams. The most common arrangement was for the dressed or squared timbers to be on the outside for appearance sake and the round logs with their bark attached "hidden" in the interior. The lengths of these beams would be cut longer than the distance centre to centre of the trestles because overlapping scarfing was generally used over each support. On top of the main beams were the three inch thick transverse deck planks (see Figs 36 & 37).

These plain beam timber bridges were built in spans of 25 feet (without corbels), 30 feet, and 35 feet. Where the span exceeded 35 feet, compound timber beams were employed in spans of 40 feet and 45 feet, and in exceptional cases up to 50 feet span; above this limit truss bridges were adopted. John A. McDonald had developed a rational method of designing shear keys and bolts in compound timber beams so that the maximum shearing stresses were fully provided for, and the beam was not excessively weakened by cutting holes for keys. The full details of his method can be obtained from, "The strength and elasticity of ironbark timber as applied to works of construction" by W. H. Warren in the Journal and Proceedings of the Royal Society of NSW.

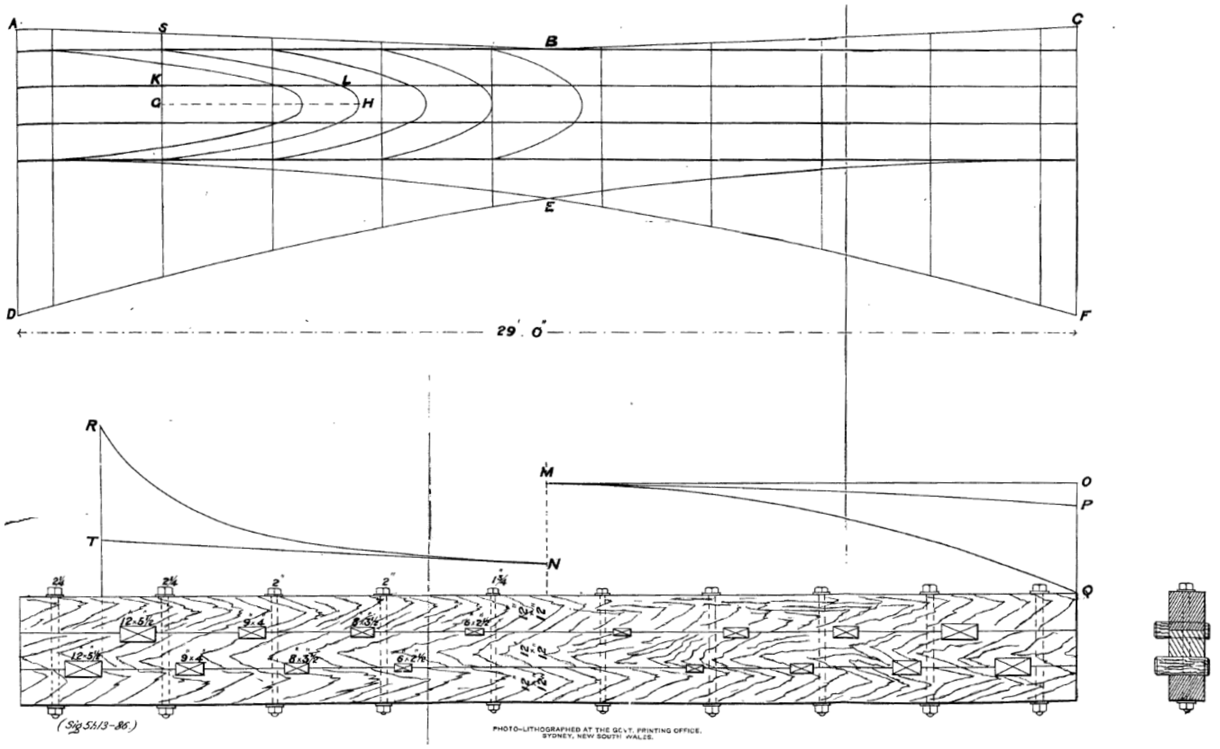


Figure 54: Diagrams provided by John A. McDonald to the Royal Society of NSW on 1st December 1886

In 1893 Percy Allan began his redesign of both the timber beam and timber truss bridges. His purpose was to simplify construction and maintenance and to minimise the cost. A costly maintenance item was the replacement of headstocks that were attached to the tops of the piles by a combination of internal mortise and tenons, and external strap bolts. The whole superstructure at that trestle had to be raised by the height of the tenon so the headstock could be raised and withdrawn. Allan introduced a pair of half-headstocks or capwales checked into the piles at their tops, one on each side, and cross-bolted. They could be replaced without raising the superstructure. The time consuming carpentry required to make the mortise and tenons, and the use of the strap bolts were eliminated. Care was taken, however, to ensure that sufficient strength and stiffness remained to face flood loads, with different designs being prepared for high level (above the highest flood) and low level bridges. Furthermore, the lengths of the corbels were reduced, scarfing of the main beams over the trestles along with the metal strap was eliminated, simple squared butt ends introduced and the number of shear keys was reduced.

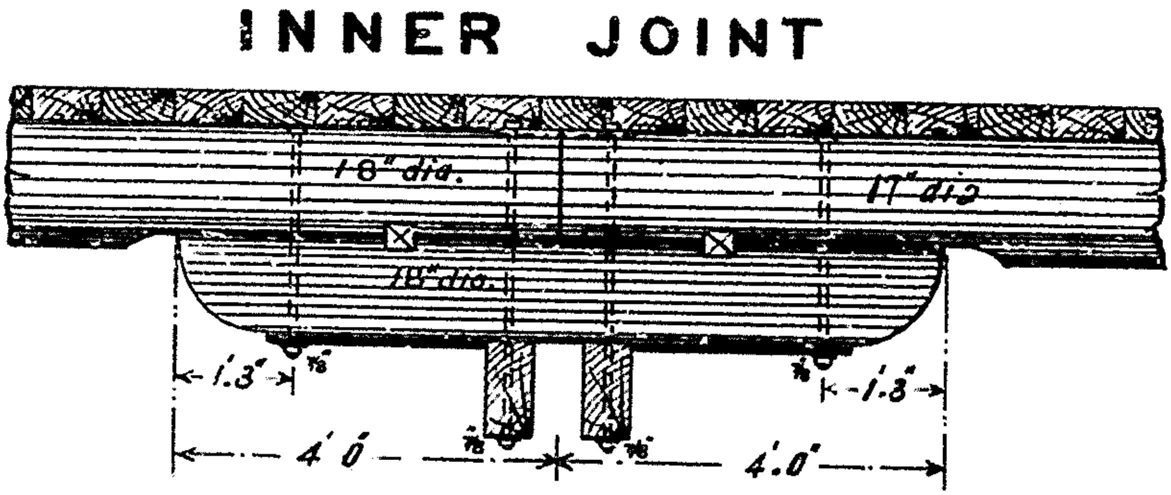


Figure 55: Post-1893 Plain Beam Bridges: short corbel, minimal shear keys, no scarf joint, two headstocks

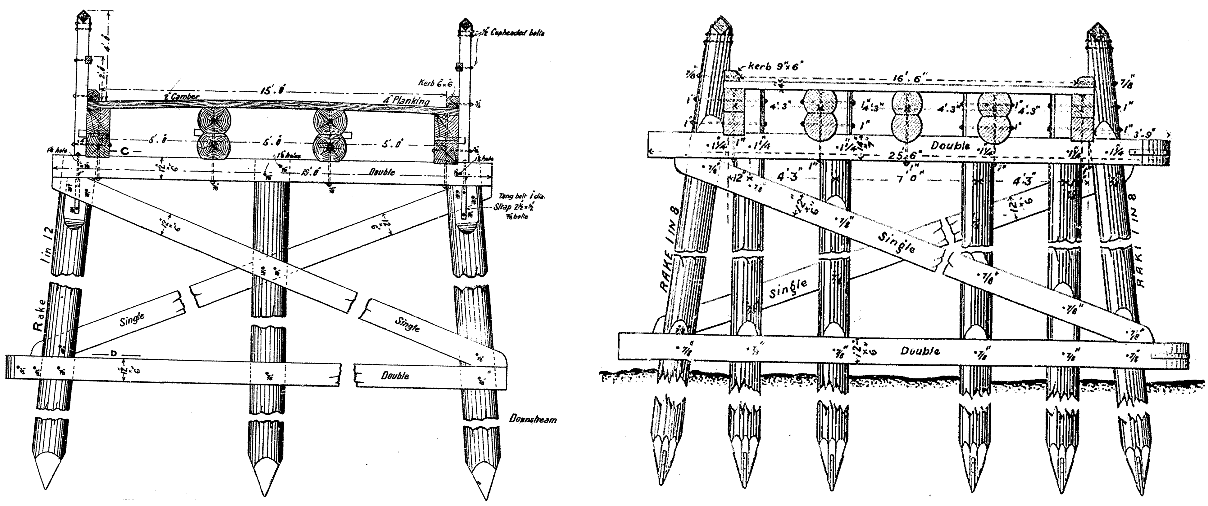


Figure 56: Post-1893 Plain Beam Bridges: high level (left) and low level showing extended piles (right)

2.10 Evolution of Road Traffic in New South Wales

The timber truss bridges of New South Wales were designed and constructed at a time when road transport looked radically different to what it does today. The roads were such that vehicles travelled much more slowly than they do today, and vehicles were much lighter than today.

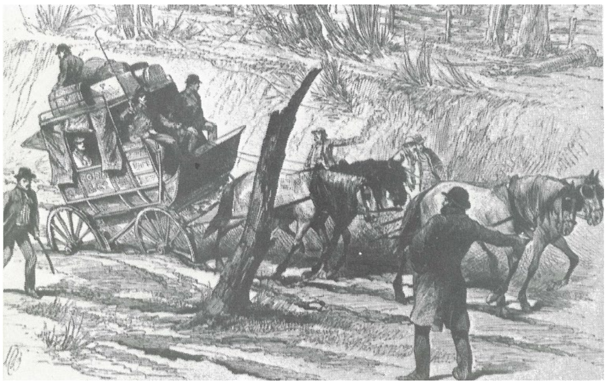


Figure 57: NSW Road Transport in the 1850s

Figure 58: Fording a River in NSW in the 1850s

The design vehicle for most timber truss bridges was a single slow moving 16 tonne traction engine, and the vast majority of the timber truss bridges were designed as single lane bridges only. The most severe load at that time was generally cattle loading which was taken as 4 kPa, and with the poor condition of most roads, no greater capacity than this was needed for the bridges.

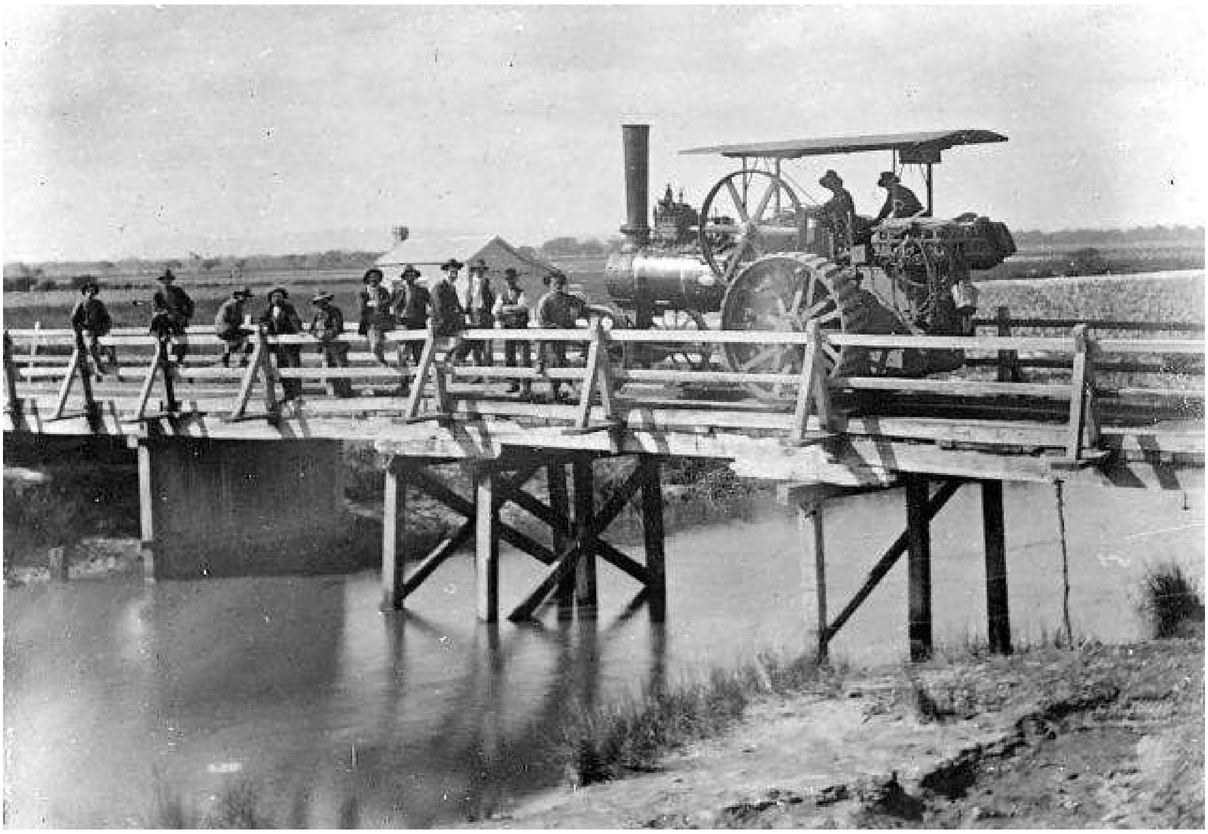


Figure 59: Photograph of a Slow Moving Traction Engine Crossing a Timber Bridge with Pedestrians

However, as roads improved, loads increased, and it was reported in the Sydney Morning Herald in January 1904: "Each harvest reports are published concerning the great loads of wheat drawing to the country railway stations. It has become the fashion to assume that each big load is the biggest or the "record"… A recent visitor to Narromine has however, furnished us with particulars of a load which was probably one of the largest ever pulled in Australia. The wagon carried 193 bags of wheat… the gross being… 24 tons 19cwb 1qr… It is anticipated that efforts will be made to beat this record, and one large grower has announced that he will send in a load of 200 bags, or break every wagon he owns." It is evident that the competition continued some time, as by 1916 the new record was 313 bags of wheat with a gross weight of almost 30 tons.

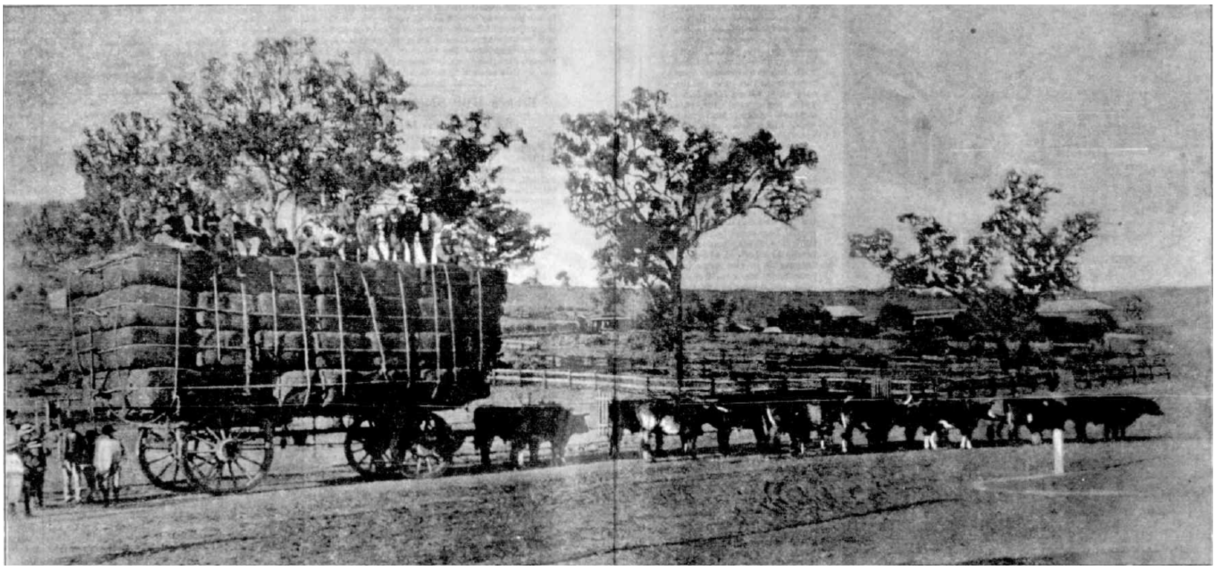


Figure 60: Vehicular Loads in Cowra in 1892, from the Town and Country Journal, June 25, 1892

Figure 61: A Great Load of Wheat: 313 Bags (over 26 tons) on a Wagon (approx 30 ton in total) in 1916.

Since 1900, the design loadings for bridges on average have increased by almost ten percent every ten years, and the actual weights of vehicles have also increased dramatically. However, the weight of heavy vehicles was not the only thing to change with time. Current Australian design rules for trucks require braking systems to be capable of decelerating at a minimum rate of approximately 0.45g. Due to the fact that this is a minimum requirement, most heavy vehicles are capable of much greater deceleration. This was not the case historically, and these decelerations impart very large horizontal forces onto bridges, for which they were not designed.

Figure 62: NSW Road Transport 100 years later

Timber bridges were generally built in the most economical form. Width, height, vertical alignment and horizontal alignment were not critical for the vehicles and speeds of 100 years ago.

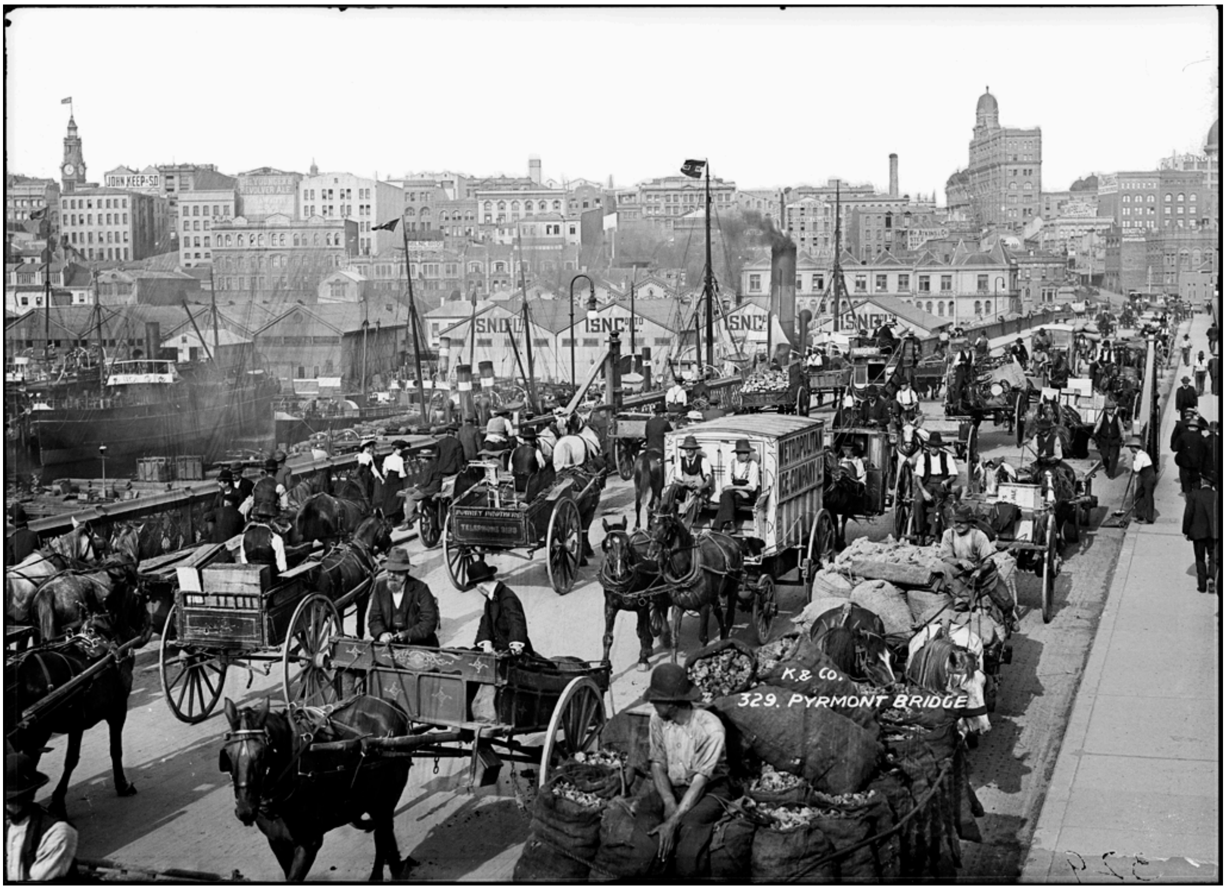


Figure 63: Photograph of traffic on Pyrmont Bridge in Sydney (Pyrmont Bridge opened in 1902)

2.11 References

2.11.1 Technical Papers and Journal Articles

- A Survey of Bridge Building in Australia, *The Australasian Engineer*, 7-11-1939
- Allan, Percy (1924), Highway Bridge Construction I-VI, *Industrial Australian and Mining Standard*, 14-08-1924, pp 243-246; 21-08-1924, pp 281-285; 28-08-1924, pp 318-322; 04-09-1924, pp 356-358; 11-09-1924, pp 394-396; 18-09-1924, pp 432-436
- Allan, Percy (1931), The George's River Bridge, New South Wales, Minutes of the Proceedings of the Institution of Civil Engineers (London), Part 2 Vol 232, 1931, pp 183-219
- Allan, Percy (1907), The Pyrmont Bridge, Sydney, NSW, Minutes of the Proceedings of the Institution of Civil Engineers (London), Part 4 Vol 170, 1907, pp 137-158
- Allan, Percy (1897), The Wagga Wagga Timber Bridge, NSW, Minutes of the Proceedings of the Institution of Civil Engineers (London), Volume 128, 1897, pp 222-225
- Allan, Percy (1895), Timber Bridge Construction in New South Wales, read before the Engineering Section of the Royal Society of NSW on 18-09-1895
- Best, R.E. & Fraser, D.J. (1982), Railway Lattice Girder Bridges in New South Wales, Conference on The Protection of Engineering Heritage, Brisbane 9-12-05-1982, pp 85-92
- Bowie, I. (2010), Australia's First Materials Testing Machine, *ASHET News*, Australian Society for History of Engineering and Technology Newsletter, Vol 3, No 4 Oct 2010, pp 4-6
- Bradfield, J.J.C. (1896), Some Notes on Australian Timbers, read before the Sydney University Engineering Society on 28-05-1896
- Bridge Building in New South Wales 1788-1938, extracts from December 1950, September 1951 and December 1954 issues of *Main Roads* (Journal of Department of Main Roads NSW)
- Dare, H.H. (1909), Preliminary Work on the Proposed Connection between Sydney and North Sydney, with some Notes on Long Span Bridges and Subaqueous Tunnelling, read before the Sydney University Engineering Society 13-10-1909
- Dare, H.H. (1900), Presidential Address, read before the Sydney University Engineering Society 9-05-1900
- Dare, H.H. (1904), Recent Road-Bridge Practice in NSW, Proceedings of the Institution of Civil Engineers (London), Vol 115, 1903-04, pp 382-400
- Dare, H.H. (1913), The Hunter River Flood of May 1913, read before the Sydney University Engineering Society 17-09-1913
- Dare, H.H. (1896), The Opening Bridges of New South Wales, read before the Sydney University Engineering Society 25-11-1896
- De Burgh, E.M. (1900), On the Use of Monier Pipes as a Pile Covering and in place of Cast-iron for Cylinder Foundations, paper no. 3200, Minutes of proceedings of the Institution of Civil Engineers (London), Vol 142 1900, pp 288-291
- Deakin, M.A.B. & Fraser, D.J. (1994), Curved-Tracked Bascule Bridges in NSW and their Relationship to the Cardioid, *Transactions of Multi-Disciplinary Engineering*, Institution of Engineers Australia, Vol. GE18 No.2 1994 pp 163-171

- Fraser D.J. (2010), American Bridges in New South Wales 1870-1932, *Australian Journal of Multi-disciplinary Engineering*, Institution of Engineers Australia, Vol. 7, No. 2, 2009 pp 109-115
- Fraser D.J. (2009), American Bridges of the NSW North Coast Railway, *Australian Journal of Multi-disciplinary Engineering*, Institution of Engineers Australia, Vol. 8, No. 1, 2010 pp 23-31
- Fraser, D.J. (1992), Cowra Bridge – Preservation of a Unique Structure, Sixth National Conference on Engineering Heritage 1992, Hobart 5 - 7-10-1992
- Fraser, D.J. (1985), Moveable Span Bridges in New South Wales Prior to 1915, *Transactions of Multi-Disciplinary Engineering*, Institution of Engineers Australia, Vol. GE9 No.2 1985 pp 71-81
- Fraser, D.J. (1986), The First Sixty Years of Metal Bridges in NSW, *Transactions of Multi-Disciplinary Engineering*, Institution of Engineers Australia, Vol. GE10 No.1 1986 pp 44-53
- Fraser, D.J. (1985), Timber Bridges of New South Wales, *Transactions of Multi-Disciplinary Engineering*, Institution of Engineers Australia, Vol. GE9 No.2 1985 pp 192-202
- Fraser D.J. (1981), Two Whipple Trusses in New South Wales – An Engineering Heritage Case Study, *Civil Engineering Transactions*, Institution of Engineers Australia 1981, pp 272-282
- Gasparini, D. (1997), American Truss Bridge Connections in the 19th Century I: 1829-1850, *Journal of Performance of Constructed Facilities*, ASCE, Vol 11. No 3, August 1997, pp 119-129
- Glencross-Grant, R. (2009), Large Road Bridges in Northern NSW: 19th Century Evolution from Timber to Iron and Back Again, *Australian Journal of Multi-disciplinary Engineering*, Institution of Engineers Australia Vol 7 No 2, 2009, pp 117-127
- King, W.K. & Fraser D.J. (1983), The Hawkesbury River Railway Bridge (1886-1946), The Engineering Conference, Newcastle 18-22-04-1983, pp 36-42
- Laws, F. (1931), Types of Highway Bridges, *Main Roads*, (Journal of Department of Main Roads, New South Wales) Volume II, No 5, January 1931 pp 69-78
- Maiden, J.H. (1889), Notes on some New South Wales Timbers, Minutes of proceedings of the Engineering Association of New South Wales 16-05-1889
- Memoirs William Christopher Bennett, Minutes of Proceedings of the Engineering Association of New South Wales, Volume IV, 1888/89, pp 216-217
- Obituary Henry Harvey Dare, *Journal of the Institution of Civil Engineers* (London), Vol 34, Issue 5, March 1950, p 103
- Obituary William Christopher Bennett, Excerpt Minutes of Proceedings of The Institution of Civil Engineers (London) Vol XCIX, Session 1889-90 Part I, pp 1-5
- Warren, W.H. & Dare, H.H. (1901), The Determination of Working Stresses for Bridges, read before the Sydney University Engineering Society 18-10-1901
- Warren, W.H. (1890), Some Applications of the Results of Testing Australian Timbers to the Design and Construction of Timber Structures, the Royal Society of NSW 3-09-1890
- Warren, W.H. (1886), The Strength and Elasticity of Ironbark Timber as applied to Works of Construction, read before the Royal Society of NSW 1-12-1886
- Warren, W.H. (1911), The Strength, Elasticity, and Other Properties of New South Wales Hardwood Timbers 8-06-1911
- Warren W.H. (1889), The Transverse Strength of Australian Timbers, Minutes of proceedings of the Engineering Association of New South Wales 16-05-1889

2.11.2 Reports to Parliament

- Martindale, B.H. (1857), Report to the Honourable the Secretary for Lands and Public Works on the Internal Communications of New South Wales, 22-10-1857
- Martindale, B.H. (1858), Second Report to the Honourable the Secretary for Lands and Public Works on the Internal Communications of New South Wales, 29-04-1858
- Martindale, B.H. (1859), Third Report to the Honourable the Secretary for Lands and Public Works on the Internal Communications of New South Wales, 1-10-1859
- Martindale, B.H. (1860), Fourth Report to the Honourable the Secretary for Public Works on the Internal Communications of New South Wales, September 1860
- Annual Reports of the Department of Public Works to the Legislative Assembly 1888-1950

2.11.3 Newspapers (Chronological Order)

- Western District, *The Sydney Morning Herald*, Saturday 20-03-1858, p 5
- Visit of Captain Martindale, *Bathurst Free Press and Mining Journal*, Wednesday 27-10-1858, p 2
- Parliamentary Paper, Great Northern Road, *The Maitland Mercury & Hunter River General Advertiser*, Tuesday 12-10-1858, p 2
- Bridge over the Paterson, *The Sydney Morning Herald*, Friday 11-03-1859, p 2
- Philosophical Society, *Empire*, Thursday 20-09-1860, p 4
- Country Works, *The Sydney Morning Herald*, Friday 21-12-1860, p 7
- Country Works, *The Sydney Morning Herald*, Monday 21-01-1861, p 5
- Births, Marriages, and Deaths, *The Sydney Morning Herald*, Saturday 20-07-1861, p 7
- Opening of the Arnold Bridge, over the River Page at Murrurundi, *The Sydney Morning Herald*, Wednesday 18-09-1861, p 10
- New South Wales Parliament, Legislative Assembly, Tuesday December 10, *The Sydney morning Herald*, Wednesday 11-12-1861, pp 3-4
- New South Wales Parliament. Legislative Assembly Monday December 16, 1861, *The Sydney Morning Herald*, Tuesday 17-12-1861, pp 3-4
- New South Wales Parliament, Legislative Assembly, Tuesday, August 12, *The Sydney Morning Herald*, Wednesday 13-08-1862, pp 3-4
- Country Works, *The Sydney Morning Herald*, Friday 21-11-1862, p 8
- Progress in Bridge Building, *The Maitland Mercury & Hunter River General Advertiser*, Saturday 21-11-1863, p 2
- Roads, Report from Commissioner, *The Sydney Morning Herald*, Tuesday 12-12-1865, p 5
- Public Opening and Naming of the Gostwyck Bridge, *The Maitland Mercury & Hunter River General Advertiser*, Saturday 3-08-1878, p 12
- The Civil Service List, *The Sydney Moring Herald*, Monday 6-04-1885, p 7
- Denman, *The Maitland Mercury & Hunter River General Advertiser*, Tuesday 2-06-1885, p 7

- The Paterson Bridge, *The Maitland Mercury & Hunter River General Advertiser*, Tuesday 9-11-1886, p 7
- New Suspension Bridge, Opening Ceremony, *The Town and Country Journal*, 28-05-1889, p 8
- Royal Society of New South Wales, Annual Meeting, *The Sydney Morning Herald*, Thursday 5-05-1892, p 9
- Cowra – A Flourishing Township in a Good Agricultural District of New South Wales, *The Town and Country Journal*, 25-06-1892, pp 26-27
- The New Bridge at Tuena, *The Town and Country Journal*, 3-11-1894, p 31
- The New Wilcannia Bridge, *The Town and Country Journal,* 21-03-1896, p 26
- Mr. E. M. de Burgh, *The Sydney Morning Herald*, Wednesday 12-11-1902, p 8
- About People, *The Town and Country Journal*, 25-09-1907, p 26
- Harbor Affairs, *The Poverty Bay Herald* (New Zealand), Tuesday 20-02-1912, p 4
- The Harbor Engineer, *The Poverty Bay Herald* (New Zealand), Thursday 11-12-1913, p 7
- Sydney Engineers Honoured, The Telford Premiums, *The Sydney Morning Herald*, Wednesday 4-05-1921, p 10
- Mr. Percy Allan, Colleagues' Farewell, *The Sydney Morning Herald*, Friday 5-03-1926, p 12
- Personal, *The Sydney Morning Herald*, Saturday 16-07-1927, p 14
- Mr E. M. de Burgh, Distinguished Career, *The Sydney Morning Herald*, Wednesday 14 -09-1927, p 16
- Mr E. M. de Burgh, Prominent Engineer's Death, *The Sydney Morning Herald*, Friday 5-04-1929, p 17
- Obituary, Mr E. M. de Burgh, *The Sydney Morning Herald*, Saturday 6-04-1929, p 21
- E. M. de Burgh, An Appreciation, *The Sydney Morning Herald*, Thursday 11-04-1929, p 10
- Mr. Percy Allan, Noted Engineer's Death, *The Sydney Morning Herald*, Thursday 8-05-1930, p 12
- Shot Through the Head: Gisborne Man's Death, Mr. John A. McDonald, *The Poverty Bay Herald*, Wednesday 4-06-1930, p 7
- Engineer Honoured, *The Sydney Morning Herald*, Friday 10-07-1931, p 12
- Sydney Harbour Bridge Supplement, *The Sydney Morning Herald*, Saturday 19-03-1932
- Mr H. H. Dare's Retirement, *The Sydney Morning Herald*, Thursday 3-01-1935, p 5
- Enquiry into Floodwaters Plan, Sydney Engineer Appointed on Outside Advice, Distinguished Career, *The Advertiser* (Adelaide), Saturday 30-03-1935, p 19
- Bridges Across the Years by J. J. C. Bradfield, *The Sydney Morning Herald* Engineering Supplement, Monday 28-03-1938, p 12s
- Deaths, *The Sydney Morning Herald,* Tuesday 23-08-1949, p 14
- Sydney's Bridges Old and New, *The Australian Women's Weekly*, Wednesday 22-04-1970, pp 77-77, 79

2.11.4 Books

- Coltheart, L. & Fraser, D.J. (1987), Landmarks in Public Works, Engineers and Their Works in New South Wales 1884-1914, Sydney: Hale & Ironmonger Pty Ltd 1897
- Department of Main Roads, NSW (1976), The Roadmakers, A History of Main Roads in New South Wales, Sydney: Department of Main Roads NSW 1976
- Department of Main Roads, NSW (1987), Timber Truss Bridge Maintenance Handbook, Sydney: Department of Main Roads NSW February 1987
- Ford, R.L., (1995), William's River, The Land and Its People, R.L. Ford 1995
- Hutchinson, F. (1896), New South Wales: the Mother Colony of the Australias, Sydney: Charles Potter, Government Printer, Phillip Street 1896
    - J.H. Maiden, Timbers of the Colony, pp 168-180
    - J.W. Holliman, Public Works, including Road and Bridges pp 287-297
- Monash University, The Centre for Continuing Education, Timber Bridges, Papers presented at a National Seminar held at Monash University on 6-8-11-1985
    - C. O'Connor, History of Timber Bridges in Australia, pp 1-23
    - E.G. Trueman, Replacement Versus Conservation, pp 341-354
- O'Connor, C. (1985), Spanning Two Centuries, Historic Bridges of Australia, Queensland: University of Queensland Press 1985
- Warren, W.H. (1894), Engineering Construction in Iron, Steel, and Timber, London: Longmans, Green & Co. and New York: 15 East 16th Street 1894

2.11.5 Miscellaneous

- Calculation Books, John A. McDonald & Percy Allan, held by RMS Bridge Engineering
- Heritage Council of NSW, Timber Truss Road Bridges of NSW: Review of Roads and Traffic Authority's Proposed Approach to Conservation, July 2011
- MBK (McMillan Britton & Kell Pty. Ltd.), Study of Relative Heritage Significance of all Timber Truss Road Bridges in NSW, December 1998
- Cardno MBK, Study of Relative Heritage Significance of RTA Controlled Timber Beam Road Bridges in NSW, October 2000; Study of Heritage Significance of Pre-1930 RTA Controlled Metal Road Bridges in NSW, February 2001
- Mackay, L.H., Timber Truss Bridges in New South Wales, Thesis, Bachelor of Architecture, University of Sydney, 1972
- RTA, Timber Truss Road Bridges: A Strategic Approach to Conservation, July 2011
- RTA, Maintaining the Links – Maintenance of Historic Timber Bridges in NSW, Oral History Program, 2001
- RTA Thematic History: A Component of the RTA Heritage and Conservation Register, 2nd Edition, 2006
- Trueman, E.G. (1984), Timber Bridge Conservation in New South Wales, Sydney: Hughes Trueman Ludlow, 1984

3 Structural Design in the Context of Heritage

3.1 The Burra Charter and Heritage Council Guidelines

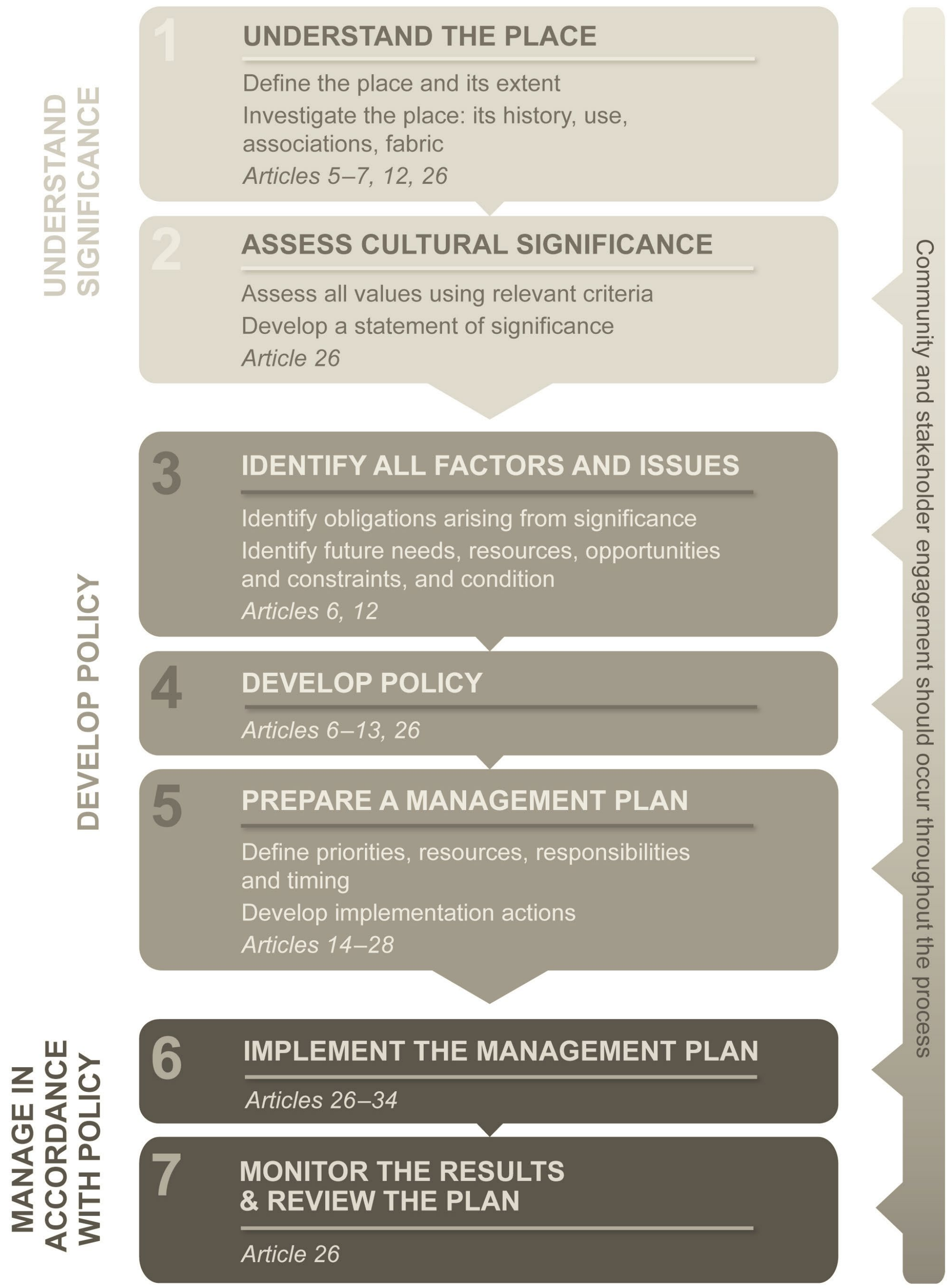
Best practice in heritage conservation is outlined in The Burra Charter: The ICOMOS Charter for Places of Cultural Significance 2013, along with the associated Practice Notes. The Burra Charter is based on the collective knowledge and experience of Australia ICOMOS members. It outlines that conservation is an integral part of the managing heritage places and is an on-going responsibility. Specifically, the Burra Charter is a set of principles that guide heritage practitioners working in Australia. Two key principles of the Burra Charter are:

- **Significance Guides Decision Making:** Understand the importance and significance of a place and its fabric before making any decisions about its future.
- **A Cautious Approach to Change:** Do as much as necessary to care for the place and make it useable, but change it as little as possible so that its cultural significance is retained.

The NSW Heritage Council and Heritage Division have mandated heritage conservation practice for NSW. The Heritage Council developed the following seven criteria gazetted under section 4A(3) of the Heritage Act 1977 to help guide decisions about whether an item is of cultural (heritage) significance, and furthermore, whether it is of State significance or of local significance. The contributions of all relevant aspects of cultural significance need to be respected in design.

- **Criterion (a) (Historical):** An item is important in the course, or pattern, of NSW's cultural or natural history (or the cultural or natural history of the local area);
- **Criterion (b) (Associational):** An item has strong or special association with the life or works of a person, or group of persons, of importance in NSW's cultural or natural history (or the cultural or natural history of the local area);
- **Criterion (c) (Aesthetic / Technical):** An item is important in demonstrating aesthetic characteristics and/or a high degree of creative or technical achievement in NSW (or the local area);
- **Criterion (d) (Social):** An item has strong or special association with a particular community or cultural group in NSW (or the local area) for social, cultural or spiritual reasons;
- **Criterion (e) (Archaeological):** An item has potential to yield information that will contribute to an understanding of NSW's cultural or natural history (or the cultural or natural history of the local area);
- **Criterion (f) (Rarity):** An item possesses uncommon, rare or endangered aspects of NSW's cultural or natural history (or the cultural or natural history of the local area);
- **Criterion (g) (Representativeness):** An item is important in demonstrating the principal characteristics of a class of NSW's cultural or natural places; or cultural or natural environments (or a class of the local area's cultural or natural places; or cultural or natural environments).

The Burra Charter includes the flowchart given below which is the Burra Charter Process. Understanding cultural significance comes first which leads to a Statement of Significance, then development of policy and finally management of the place in accordance with the policy. This requires a wide range of knowledge, skills and disciplines. Changes in circumstances, or new information or perspectives, may require reiteration of part or all of the Burra Charter Process.



3.2 ICOMOS Principles for Historic Timber Structures

ICOMOS (International Council on Monuments and Sites) is a global non-government organisation working for the conservation and protection of cultural heritage. Australia ICOMOS is the national Australian chapter. At the 12th general assembly it adopted the "Principles for the Preservation of Historic Timber Structures (1999)". This document makes a number of helpful recommendations. Some relevant principles include:

- *"The primary aim of preservation and conservation is to maintain the historical authenticity and integrity of the cultural heritage. Each intervention should therefore be based on proper studies and assessments. Problems should be solved according to relevant conditions and needs with due respect for the aesthetic and historical values, and the physical integrity of the historic structure or site."*

- *"Any proposed intervention should for preference:*
  follow traditional means;
  be reversible, if technically possible; or
  at least not prejudice or impede future preservation work whenever this may become necessary; and
  not hinder the possibility of later access to evidence incorporated in the structure."

- *"The minimum intervention in the fabric of a historic timber structure is an ideal. In certain circumstances, minimum intervention can mean that their preservation and conservation may require the complete or partial dismantling and subsequent reassembly in order to allow for the repair of timber structures."*

- *"The aim of restoration is to conserve the historic structure and its load-bearing function and to reveal its cultural values by improving the legibility of its historical integrity, its earlier state and design within the limits of existing historic material evidence, as indicated in articles 9 - 13 of the Venice Charter. Removed members and other components of the historic structure should be catalogued, and characteristic samples kept in permanent storage as part of the documentation."*

- *"In the repair of a historic structure, replacement timber can be used with due respect to relevant historical and aesthetical values, and where it is an appropriate response to the need to replace decayed or damaged members or their parts, or to the requirements of restoration."*

- *"It should be accepted that new members or parts of members will be distinguishable from the existing ones. To copy the natural decay or deformation of the replaced members or parts is not desirable. Appropriate traditional or well-tested modern methods may be used to match the colouring of the old and the new with due regard that this will not harm or degrade the surface of the wooden member."*

- *"Contemporary materials, such as epoxy resins, and techniques, such as structural steel reinforcement, should be chosen and used with the greatest caution, and only in cases where the durability and structural behaviour of the materials and construction techniques have been satisfactorily proven over a sufficiently long period of time."*

3.3 Attitudes to Conservation

As stated by Sir Henry Maybury, of the Institution of Municipal and Country Engineers in 1923, "*It sometimes falls to your lot to deal with the old structures in the country which we have inherited, which have been the work of those who have preceded us and have 'builded better than they knew'. It often happens that these bridges are not sufficiently strong and commodious to support present-day traffic. Do not let it be said that we have had no regard for art for art's sake, that we have carried out some work which has despoiled some glorious old bridge which we have inherited, and thus in our day been guilty of an act of desecration. Oftentimes with care the necessary strength and width can be acquired without materially affecting the beauty of the old structure.*"

In 1924, the editor of the *Industrial Australian and Mining Standard* wrote as an introduction to a series of articles by Percy Allan, "*The conditions to be met in satisfying the requirements calling for the construction of a bridge vary widely, according to those conditions and the natural features on site… But the subject has an additional charm, in that the bridge building art calls for the exercise of much original thought in the choice of materials available locally, and their adaptation to the work in hand.*" The timber truss bridges of New South Wales show an evolution in understanding of timber. This understanding came through experiments, study and experience. However, due to the reduced stock of timber, and the increased affordability of steel and concrete in the 1900s, interest in and understanding of timber has been on the decline. In 1896, J.H. Maiden, Superintendent of Technical Education wrote, "*Ironbark stands alone as the embodiment of the combination of a number of qualities valued in timber, viz., hardness, strength, and durability… one of the main reasons why colonial timbers are not more used is because users are nervous through ignorance… I plead for a wider interest to be taken in our trees and our timbers, and that in place of the apathy which exists… it may be realised that study of them is not only full of interest, but, as a mental discipline alone, worthy of attention by the best intellects of the Colony.*" In 1932, William Atwood said, "*This generation of engineers may well ask themselves whether in their frequent disregard of the oldest and one of the best of the available structural materials, they are worthily 'carrying the torch of progress'.*"

The original designers cared not only about economy, but also about ecology, and about aesthetics. Bennett once wrote in the margin of a minute by one of the road superintendents, who had recommended the construction of a bridge over the Hunter, where bridges were in those days much rarer than they are now, "What! Another bridge over the Hunter! The river will soon be roofed in, and there will be no space for the cattle to drink." The fact that the designers cared about appearance is clear from the fact that they always specified that the outer girders and corbels of the approach spans should be cut square, and only the hidden girders remain rounded.

In order to conserve these bridges, we need to apply our engineering intellects to understanding the structural behaviour of timber and how it can carry today's loads. Experience has shown that these structures cannot be preserved by removing vehicular traffic (they deteriorate very quickly when used only for pedestrian traffic), so they must be strengthened in order to carry today's traffic loads. In order to do this, we need to respect timber as a structural material, and not just as "fabric" in the bridge which needs to be "preserved". We also need to respect the original designers, who were a group of internationally renowned engineers working collaboratively, building on each other's expertise and ideas, to design a group of bridges which reflect engineering and technical excellence, shown by the fact that they have survived until this day.

3.4 Conservation of Bridges and Conservation of Forests

As of January 2014, there were 19 listed World Heritage Places in Australia, 16 of which are natural heritage places such as rainforests, wilderness areas and national parks. Similarly, a large proportion of the places on the National Heritage List consist of natural rather than built heritage. This indicates the importance of conservation of natural as well as built heritage.

Figure 64: Wet Sclerophyll Forest, Lamington National Park, Lismore – World Heritage Area

The availability of high-quality hardwood timber required for heritage timber truss bridges is a substantial concern. It is an increasingly scarce resource, and is valuable as part of our natural heritage, as well as for its usefulness in carrying heavy vehicles over heritage timber truss bridges.

The key timbers required have to be derived from 80 to 200 year old trees in order to achieve the necessary strength, durability and dimensions. The major structural elements require species from old-growth forests that are often rare outside national parks. These bridges need species such as Grey Box, Ironbark, Tallowwood and Grey Gum, as used in the original designs. Lesser timbers such as Blackbutt or Spotted Gum are inferior, have less strength and deteriorate at a faster rate, thus requiring more frequent replacement. Searches have been conducted Australia-wide which have failed to yield sufficient timber of suitable quality and dimensions.

Although at least two pieces of heart-free sap-free bridge timber should be able to be recovered from a single log, in practice (on average) less than a single piece per log meets the requirements for use in most heritage timber truss bridges. This increases the responsibility of designers to ensure that designs maximise the durability of timber in order to minimise the need to cut down old growth forests, which are part of the valuable natural heritage of this country. Careful consideration should also be given to replacing timber with modern materials where the heritage significance of the fabric of the particular element is low or moderate, and where the introduction of modern materials would not substantially affect the heritage significance of the whole bridge.

3.5 Application of Heritage Principles to Structural Design

3.5.1 Community Focused Design

The best way to conserve a heritage structure is to ensure that the local community values it.

There is never a shortage of people wanting to rid the roads of old timber bridges. There are those who think the timber bridge an ignoble structure. When the London historian William Maitland heard in 1736 that the London Bridge was to be a timber structure, he declared: "*upon these, and two end piers, 'tis said will be erected thirteen wooden arches! Which will not only greatly redound to the dishonour of the nobility, gentry, &c., in these parts, for whose convenience it is chiefly intended, but likewise to the kingdom in general, to have a disgraceful wooden bridge erected so near its capital city, …for nothing looks more mean and beggarly than a wooden bridge…*" There are pragmatists who think timber bridges impractical. The newspaper reporting the opening of Gostwyck Bridge (an Old PWD truss) in 1878 had the following to say: "*But we cannot compliment the department on a very vigorous display of sagacity. The bridge is a long bridge… It is not wide enough to allow two vehicles safely to pass each other. When a bridge is being erected to accommodate traffic, why in the name of common sense should it not be made to accommodate traffic without danger and trouble? … It must not be supposed that Gostwyck Bridge is more unsafe than other bridges: they are all in the same plight, and of that we complain.*" There are those who fear the clickety clack of timber decks as they cross these old bridges, thinking the bridge near to collapse.

A bridge that looks like it is the result of band-aid solutions, left to deteriorate until traffic restrictions are put in place to carry out repairs is less likely to be valued by the community. Poor design of strengthening work, poor construction and poor maintenance accentuate this problem. If the owner of the asset does not take pride in it, the local community are unlikely to value it.

A community is more likely to value a structure if it has an element of beauty or elegance to it. It is also more likely to value a structure if convenience is maximised and inconvenience minimised. Community sentiment can also be assisted by education. Providing information regarding the history and the ingenuity of the original design and its place in New South Wales may assist. A community focused design will include, but not be limited to, the following:

- **Elegance in Design:** The bridge, and any additions to it, should be in keeping with the elegance and simplicity of the original design (see also RMS, Bridge Aesthetics, 2012);
- **Road Safety:** The design should be safe for vehicles and for pedestrians where appropriate. This will require sensitively upgraded barrier rails, alignments and approach treatments;
- **Transparency in Design:** Design should enable the inquisitive to determine the original details, fabric and form where possible by not obscuring this by changes and additions;
- **Durability in Design:** The design should be detailed to maximise service life so that community impact of traffic diversions due to bridge closures is minimised;
- **Strength for Modern Vehicles:** The bridge should be strengthened to carry today's vehicles so that inconvenient load restrictions are minimised, and community benefit maximised;
- **Interpretation:** Information on the bridge and its history should be made readily available, and where appropriate, included in the vicinity of the bridge.

3.5.2 *Safeguarding of Structures*

Article 2.4 of the Burra Charter states that, "Places of cultural significance should be safeguarded and not put at risk or left in a vulnerable state". Whatever an engineer may do, it should not make the bridges more vulnerable, less robust, or at a higher risk than before. These bridges were designed by a team of engineers, all exceptional in their own right, and working together sharing knowledge and building on each other's strengths. Many of these bridges also have stood the test of time, with very few failing due to structural overload. Current day engineers should not assume they can improve on these designs just because they are using more modern materials or advanced analysis methods. The use of good materials must be matched by excellence in engineering design and detailing as well as quality construction if a safe and durable result is to be achieved. This requires an understanding of original materials and original structural behaviour.

3.5.3 *Respect for Significance*

Article 3.1 of the Burra Charter states that, "Conservation is based on a respect for the existing fabric, use, associations and meanings. It requires a cautious approach of changing as much as necessary but as little as possible". Respect for the associations with distinguished engineers implies that every effort should be made to apply engineering and technical excellence to measures that are taken to strengthen or improve these structures. Respect for the existing fabric must mean more than merely keeping some timber in the final structure. It should extend to using all materials in a way that shows a proper understanding of the strengths and weaknesses of timber as an engineering material. Many of these timber bridges are in rural settings, and the use of natural materials means that historically, the bridges have tended to harmonise with the rural landscape. Introduction of new materials must be done with discretion and sensitivity.

Sometimes the preferred outcomes for a bridge on its individual merits are different to its values as part of a diverse population. Good conservation practice recognises both these aspects.

At the very core of the conservation process is the principle that the cultural significance of a place and its fabric must be understood before any decisions are made concerning its future. This means that it is impossible to design a heritage sympathetic design without some understanding of the cultural significance of the particular bridge. Generally, a Statement of Significance for the bridge can be obtained from either the relevant government agency's Section 170 Heritage and Conservation Register or from the State Heritage Register. For many bridges, conservation management documents have been prepared which include policies, options and strategies for conservation. These documents should be regularly revised and updated because the cultural significance of a place may change as a result of the continuing history of the place, and also the understanding of cultural significance may change as a result of new information.

Some elements have particular significance because of the age of their fabric (eg, original cast iron or masonry piers, especially if more than one bridge has been built on those piers) whereas others have significance because of the detail of their design (eg, laminated timber bottom chords).

3.5.4 Transparency in Design

Article 3.2 of the Burra Charter states that, "Changes to a place should not distort the physical or other evidence it provides, nor be based on conjecture." Similarly, the ICOMOS principles as mentioned in Section 3.2 emphasise the importance of authenticity and load-bearing function. While discretion and sensitivity are critical when modern additions are provided, any strengthening works should be able to be interpreted as such, and the original design intent should not be obscured in the process. Care must be taken to first understand the cultural significance so that conflicting requirements can be balanced to give the best conservation outcome for a particular element of a bridge. Sometimes, this might mean a change in fabric or aesthetics in order to retain important physical evidence of design, and sometimes it may mean obscuring load bearing function in order to retain important fabric or aesthetic features.

3.5.5 New Work: Views To and From the Bridge

Article 22.1 of the Burra Charter states that, "New work such as additions or other changes to the place may be acceptable where it respects and does not distort or obscure the cultural significance of the place, or detract from its interpretation and appreciation." New work would generally include such things as installation of new traffic barriers and modifications to approach spans. As well as considering how and from what angles the bridge and its various elements will be viewed, consideration must be given to the overall form, bulk, scale and fabric of the rehabilitated bridge. Care must be taken before introducing new fabric or changing the sizes of elements (even elements of lower significance) to ensure that this does not negatively impact the views to and from the bridge. For elements of lower cultural significance (eg, approach span piers) it may be most appropriate to change the fabric (eg, use steel rather than timber) in order to retain the simplicity of form rather than increasing the bulk and providing double trestle piers (which not only add to the bulk and change the form, but also decrease the waterway area in times of flood). Similarly, while it is important to retain a rhythm in the barrier posts, it may be preferable to increase the post spacing when post sizes are increased so that the change in bulk of the upgraded barrier posts is minimised, and so that views are not unnecessarily obscured.

3.5.6 Appropriate Use of Modern Techniques

Article 4.2 with associated note in the Burra Charter states that, "Traditional techniques and materials are preferred for the conservation of significant fabric. In some circumstances modern techniques and materials which offer substantial conservation benefits may be appropriate… The use of modern materials and techniques must be supported by firm scientific evidence or by a body of experience." When substantial conservation benefits can be achieved, the introduction of modern materials and techniques is welcomed – such benefits may include strengthening of a structure in order to conserve its use, or protecting a structure by upgrading barrier rails to reduce the risk of collision, or introducing new durability initiatives to protect the timber fabric.

3.6 References

- Austroads, Guide to Heritage Bridge Management, 2001
- Heritage Act 1977, http://www.austlii.edu.au/au/legis/nsw/consol_act/ha197786/
- Heritage Division of the Office of Environment and Heritage NSW, State Agency Heritage Guide, Management of Heritage Assets by NSW Government Agencies, Part 1: State Owned Heritage Management Principles; and Part 2: Heritage Asset Management Guidelines, 2005 http://www.environment.nsw.gov.au/resources/heritagebranch/heritage/StateAgencyHeritageGuide.pdf
- Heritage Division of the Office of Environment and Heritage NSW, Assessing Heritage Significance, NSW Heritage Manual, NSW Heritage Office, 2001 http://www.environment.nsw.gov.au/resources/heritagebranch/heritage/listings/assessingheritagesignificance.pdf
- ICOMOS, The Burra Charter, The Australia ICOMOS Charter for Places of Cultural Significance, 2013, http://australia.icomos.org/publications/charters/
- ICOMOS, Practice Note: Understanding and assessing cultural significance http://australia.icomos.org/wp-content/uploads/Practice-Note_Understanding-and-assessing-cultural-significance.pdf
- ICOMOS, Practice Note: Developing Policy http://australia.icomos.org/wp-content/uploads/Practice-Note_Developing-Policy.pdf
- ICOMOS, Practice Note: Preparing Studies and Reports – Contractual and Ethical Issues http://australia.icomos.org/wp-content/uploads/Practice-Note_Preparing-studies-and-reports-contractual-and-ethical-issues.pdf
- ICOMOS, Practice Note: The Burra Charter and Archaeological Practice http://australia.icomos.org/wp-content/uploads/Practice-Note_The-Burra-Charter-and-Archaeological-Practice.pdf
- ICOMOS, Practice Note: The Burra Charter and Indigenous Cultural Heritage Management http://australia.icomos.org/wp-content/uploads/Practice-Note_The-Burra-Charter-and-Indigenous-Cultural-Heritage-Management.pdf
- ICOMOS, Practice Note: Interpretation http://australia.icomos.org/wp-content/uploads/Practice-Note_Interpretation.pdf
- ICOMOS, Practice Note: Burra Charter Article 22 – New Work http://australia.icomos.org/wp-content/uploads/Practice-Note_Burra-Charter-Article-22-New-Work.pdf
- ICOMOS, Principles for the Preservation of Historic Timber Structures (1999) http://www.international.icomos.org/charters/wood_e.pdf
- RMS Centre for Urban Design, Bridge Aesthetics: Design guideline to improve the appearance of bridges in NSW, July 2012
- RMS Section 170 Heritage and Conservation Register http://www.rms.nsw.gov.au/environment/heritage/heritageconservreg/index.html
- UNESCO, World Heritage List http://whc.unesco.org/en/list/

4 Structural Modelling of Timber Bridges

4.1 Understanding Timber as a Structural Material

As stated by Dykes of the British Institution of Structural Engineers in 1976, "*Engineering is the art of modelling materials we do not wholly understand, into shapes we cannot precisely analyse so as to withstand forces we cannot properly assess, in such a way that the public has no reason to suspect the extent of our ignorance.*"

This quotation is especially relevant in timber design. This section outlines the peculiarities of timber as a structural material, focusing first on the material properties (which we do not wholly understand), then the section properties (which we cannot precisely analyse) and finally the forces (which cannot be properly assessed). We can then move forward to design and assessment.

4.1.1 "Materials we do Not Wholly Understand"

Timber is different to most other materials that structural engineers use, simply because it is organic. Unlike steel and concrete, where there is a high level of control in the composition of the material (e.g. percentage carbon in steel, and specification of admixtures in concrete), timber comes as it grows, and every piece of timber is different. Even along its length, a single piece of timber has significant variations in properties. Timber is highly anisotropic, being significantly weaker when loaded across the grain rather than along it. Timber also has a tendency to creep under sustained load, and to shrink under changing moisture conditions. However, the timber used in timber truss bridges has a higher strength to weight ratio than either steel or concrete.

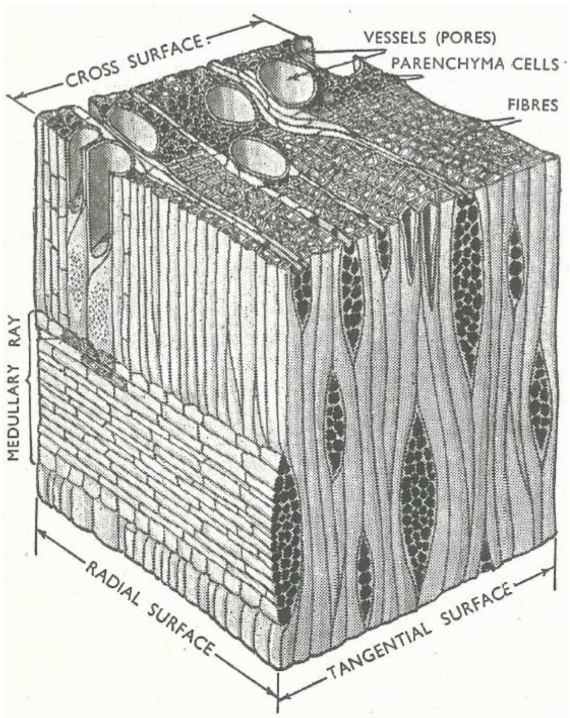


Figure 65: Characteristics of Hardwoods

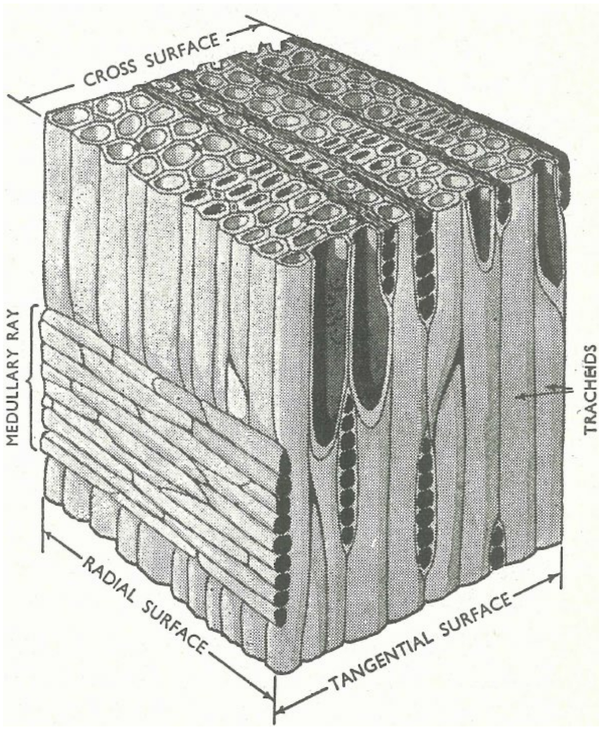


Figure 66: Characteristics of Softwoods

Timbers are generally categorised into hardwoods and softwoods. Some hardwoods are actually quite soft, while some softwoods are comparatively hard. The categorisation is based on the cell structure and the presence of 'vessels' or 'pores', rather than the hardness of the timber. Because of its cell structure, timber differs in its strength, stiffness and shrinkage properties in the three directions corresponding to the radial, tangential and longitudinal directions of the tree.

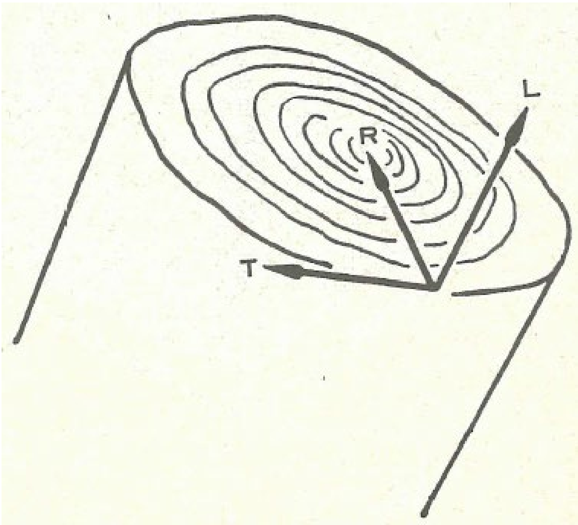


Figure 67: Principal Axes in Timber (L, R, T)

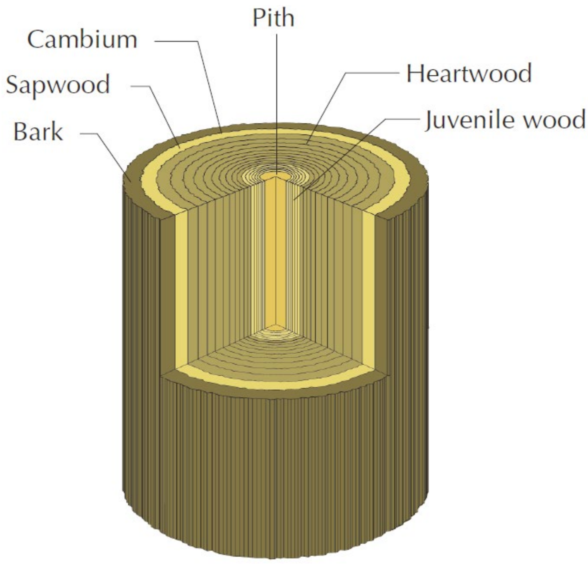


Figure 68: Cross Section – Parts of the Tree

The wood of the tree is usually differentiated into two distinct zones, the outer, sapwood and the inner, heartwood. The sapwood is generally lighter in colour than the heartwood, and although it has the same strength as heartwood, it does not have the same durability. Toxic treatments that can be given to sapwood to improve its durability are increasingly being limited by legislation.

The timbers used in bridges in New South Wales are generally limited to eucalypt and corymbia hardwood species, and the timbers used in trusses are generally limited to a few eucalypts. For replacement of members in timber bridges, species are specified in RMS specification 2380.

Density, odour and the colour of the heartwood are valuable aids in identification of a species of timber, but generally a microscopic examination of a very thin cross-section of the wood is relied upon. This is sometimes insufficient to distinguish between closely related species and so it becomes necessary to obtain for examination the leaves, fruit and bark of the tree to identify it.

There are many factors that affect the strength and stiffness of a particular piece of timber:

- **Species:** The strength properties of different species of wood vary considerably, some species being many times stronger than others. Even within a particular species, there is a wide variation in the properties of the wood, with some pieces being more than twice as strong as others in the same species of hardwood, and even wider variation in softwoods.
- **Density:** In general, the higher the density, the higher the strength is likely to be. However, the relation is not close enough for strength to be accurately predicted by measuring density.

- **Position in the Tree:** The way in which the strength properties vary with height in the tree depends on the species. In softwoods, the wood near the butt of the tree is usually somewhat denser, stronger and harder, but in some of the eucalypts the reverse holds true.
- **Conditions of Growth:** Environmental factors affecting tree growth, such as height above sea-level, temperature, rainfall, type of soil, and spacing between trees all have an effect on the strength properties of the wood, probably mainly through their effect on density.
- **Sloping Grain:** A slope in the grain of 1 in 20 has a negligible effect on the strength of a beam, 1 in 16 reduces it by about 20%, 1 in 12 by 30%, 1 in 10 by 40%, and 1 in 8 by 50%.
- **Strength Reducing Characteristics:** The presence of a knot in the timber may reduce the strength to practically zero, irrespective of the species, moisture content, etc. The extent of the influence of knots depends on their size, shape and location in the piece of timber, and the type of stress to which they are subjected. Knots are particularly harmful when subjected to tension stresses, and also have a serious effect on the stiffness of columns.
- **Shakes, Splits and Checks:** Shakes, splits and checks reduce the resistance to shear stresses, so they should be avoided at joints, and other areas subjected to high shear stress.
- **Crookedness:** Any form of crookedness is likely to cause inconvenience during construction, but it may also result in eccentricity of load, which is critical in columns.
- **Temperature:** Above normal temperatures tend to lower the strength properties of wood and lower temperatures make it stronger. The magnitude of the effect will be related to the moisture content of the wood and the period of exposure to the abnormal conditions.
- **Fire:** Large sections of hardwood timber have low flammability because they char slowly, forming a layer of charcoal which protects the wood and slows the rate of combustion.
- **Decay and Biological Attack:** Seriously decayed wood has very little strength. The three main agents of wood deterioration are fungal rot, borers and termites. The weakening effect of insect damage may vary from negligible proportions to complete destruction.
- **Moisture Content:** The moisture content of wood has a most important effect on its strength, stiffness and stability. With a drop in moisture content from 25% to 12% (fully seasoned), a piece of timber's modulus of rupture and compression strength parallel to the grain may have improved by 75-100%, and its stiffness (modulus of elasticity), will be less affected but it may have increased by up to 30%. For timber in bridges, where the minimum dimension is often greater than 100mm, it is rare that the moisture content would drop below 20%, and even if it does, the drying process is often accompanied by the formation of checks, splits and honeycombing, which largely offset the increase in strength resulting from drying.
- **Duration of Load:** The mechanical properties of wood are considerably affected by the duration of loading. The shorter the time under load the higher the strength. Repeated applications of a temporary load have a cumulative effect so that a load of relatively short duration, if continually re-applied, may have much the same ultimate effect on the structure as if it were a loading of long duration. Duration of load affects stiffness as well as strength.
- **Size:** Due to the random distribution of strength reducing characteristics in any piece of timber, there is a higher probability of finding more such features in critical areas in a member of large cross sectional area, and so average characteristic strength reduces with size.
- **Fatigue:** When timber is subject to fatigue, tests have shown that under ten million cycles of completely reversed stresses, the modulus of rupture of most species reduces by up to 75%.

4.1.2 "Shapes we Cannot Precisely Analyse"

Just as the properties of the timber as a material are subject to much variability, so is the shape of the timber. Variability applies both to the cross-sectional dimensions of the timber, and to the straightness of a member along its length. Both of these aspects are subject to change with time.

- **Timber Shrinkage:** Moisture content in timber is defined as the weight of water divided by the weight of dry wood. The moisture content of wood in a tree can often exceed 100%. When a log is sawn into lengths of timber for structural elements, the timber loses moisture to be in equilibrium with atmospheric moisture (equilibrium moisture content – usually around 20% in bridges). When the timber is first cut, the initial reduction in moisture content is a result of free water loss (until moisture content is approximately 25%), and usually occurs without any significant dimensional changes to the timber. For timber to be classed as seasoned, the moisture content must be further reduced to 15%, resulting in a reduction in the size of the cell walls, which causes the timber to shrink in size. The extent of shrinkage in timber is most pronounced in the tangential direction (perpendicular to the grain, and tangential to the growth rings), and least pronounced longitudinally or parallel to the grain. The shrinkage of timber along the grain is small, and for most purposes may be neglected. However, the shrinkage perpendicular to the grain may be in excess of 10% for timbers used in bridges, and so particular attention must be paid especially to connections.
- **Out of Straightness:** Trees are naturally prestressed structures, and timber contains residual stresses which affect its behaviour when it is cut into structural elements. Hardwood trees have compression on the inside of the trunk and tension on the newer wood towards the outside of the trunk. The stresses in the wood are released when the trunk is cut into lengths of sawn timber. If there is a residual stress differential across a piece of timber this may lead to twist, cup, bow or spring in the piece. The movement of the timber is accentuated by moisture movement, so the deformation becomes even more apparent as the wood dries.
- **Creep in Bending:** Creep in a member subject to bending occurs due to the inelastic shortening of cells on the compression side of the member. The sum of these microscopic movements can contribute to substantial movement in the member. Recoverable creep is time-dependent deflection that upon release of the load will be fully recovered. It is associated with the squashing of the timber fibres. As the fibres squash, the crystalline structure of the fibres is rearranged. Irrecoverable creep is time-dependent deflection that is not recovered when the load is released. There is microscopic damage to the fibre structure so that the load paths change and there is no stress on the structure to encourage it to return to its former configuration. Both types of creep are accelerated by moisture movements.
- **Creep Buckling:** Columns are generally designed for compressive loads, and slender columns must be checked for buckling strength. The buckling strength of a timber column is a function of a great number of complex parameters. Because timber members are never truly straight along their whole length, and because loads in timber are never truly concentric (due to material variability within the timber section), columns must be designed for combined bending and compression. Furthermore, the initial out of straightness is likely to increase due to creep, which will, in turn, increase the bending moment and hence allow more deflection. In some cases, this may lead to premature buckling due to creep, caused by the duration of the load (and its effect on stiffness) rather than the magnitude of the load. Creep, therefore, must be considered for strength of timber columns, not just serviceability.

Due to the abovementioned factors, it must be admitted that there is considerable uncertainty regarding the shape of a timber member, both in cross section and along its length. In addition to the changing properties of timber due to shrinkage, out of straightness and creep, there is uncertainty in the supply of timber. Because timber is grown, not made, the size of a timber member may very considerably. The variation in round timbers used for girders and piers can be up to 100s of millimetres at mid-length, and then there is a variation along the length due to taper. The variation in sawn timbers is usually less, but can still exceed 10%. According to RMS Specification 2380, bridge timber for truss components should be ordered rounded up to the nearest 5mm and with a 5% shrinkage allowance added to both the design thickness and width.

As well as uncertainty in dimensions, there is a random distribution of strength reducing characteristics in each piece of timber, whether this is a hollowed out section in a round girder, a series of splits and checks in a corbel, or a knot hidden in a timber truss member, all of these have an effect on the strength, stiffness, and behaviour of a structural member in a timber bridge.

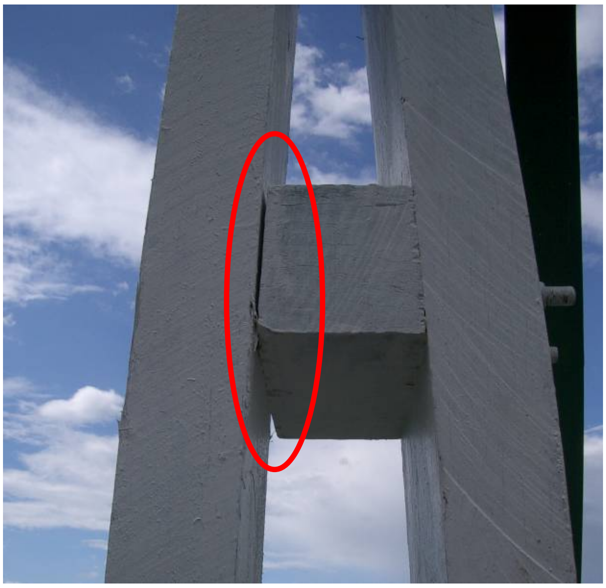


Figure 69: Tabulam Bridge (de Burgh) – difficulty fitting spacer due to cup of left-hand timber flitch

Figure 70: Clarence Town Bridge (Old PWD) – bottom chord notched on side to fit splice plate

Figure 71: Rossi Bridge (Allan Truss) – bottom chord is not square due to removal of sapwood

Figure 72: Carrathool Bridge (substructure) showing gap in connection due to shrinkage

4.1.3 *"Forces we Cannot Properly Assess"*

The purpose of a structure is to support the loads for which it has been designed. To accomplish this it must be able to transmit a load from one point to another. Structures are complex and must be idealised or simplified into a form that can be analysed. It is possible that a structure will require different idealisations under different loads. As well as idealising structures, we also idealise loads (we don't really expect a T44 vehicle to cross a bridge – it is simply an idealisation).

There are a broad range of factors which make forces in timber bridges hard to properly assess:

- **Dead Loads:** The self weight of a timber bridge is significantly more difficult to determine than the self weight of most other types of bridges. This is firstly because the density of timber is highly variable, and secondly because the cross sectional area of timber is also variable. In addition to this, the weight may change with changes in moisture content.
- **Internal Stresses:** Trees have internal stresses when they are growing, and timber used for structural purposes still contains residual stresses, which are very difficult to quantify.
- **Variable Stiffness:** When trying to determine the design bending moment in a timber girder, it has to be realised that, although the drawings might specify five F27 round timber girders with mid-span diameter of 480 mm, this does not mean that on site each girder is perfectly round with a diameter of exactly 480 mm and a modulus of elasticity of exactly 18,500 MPa. It is likely that the mid-span diameters will vary from 450 mm to 550 mm (with randomly distributed strength-reducing characteristics such as knots, hollows and checks) and the modulus of elasticity will vary from 11,000 MPa to 26,000 MPa. In a system of parallel girders with a deck on top, stiffer girders will attract higher loads. Since the two primary factors contributing to stiffness (section shape and modulus of elasticity) are so variable, it is impossible to accurately assess the real bending moment being experienced by a single girder.
- **Corbel Effects:** The contribution of the corbel toward the bending capacity of timber girders is a much debated issue. It is clear that they do contribute some level of continuity over the support between two girder spans. However, the connectivity is limited by the tightness of the bolts, the capacity of the timber in bearing under the bolt washers, and the condition of the corbel, as corbels are often subject to significant checking and sometimes splitting. All these aspects must be understood before any continuity can be assumed.
- **Deck Connections:** It is critical to be familiar with the connections likely to be found on timber decks. For the most part, timber decking is at best sporadically attached to the girders or stringers on which it sits. To assume that horizontal loads can be transferred from the deck into the girders or stringers is not likely to reflect reality. Similarly, to assume any form of composite action between the decking and the sheeting or between the deck system and the girders or stringers is unrealistic. A proper understanding of the stiffness and connectivity of the deck is critical as this affects the way loads are distributed onto the truss. It may be valid to assume that the timber sheeting (if continuous for the length of the bridge) is able to transfer the braking loads into the abutment. However, it is utterly impossible for the sheeting to take tension (thereby sharing the load with the bottom chord of a truss).
- **Load Distribution:** The load distribution between the two flitches that make up many members of timber truss bridges is highly variable. Sometimes loads are equally shared between the two flitches, and other times up to 70% of the load is taken by a single flitch.

- **Connection Capacities:** Connections are generally the weakest links in timber structures, and so before it is assumed that a particular member is carrying a particular load, it must be checked that the connections at each end of the member have the capacity to transfer the load. When connections are checked, it is not generally the metal connector that is critical, but the timber. The capacity of a connection in timber is highly dependant upon the direction of grain in the timber. While the behaviour of timber can be fairly complicated, the behaviour of timber connections is even more entangled, and has often not been subjected to rational engineering enquiry. This is currently under investigation throughout the world. It is important to realise that the values in current codes and standards are, at best, 'rules of thumb'. Trusses are generally modelled with 'compression only' and 'tension only' elements, and much care needs to be taken in detailing connections of bracing in timber piers.
- **Truss Connections:** Connections between members in trusses generally behave as pinned connections, and care must be must be taken before any moment restraint is assumed. Connections between principals and butting blocks also generally behave as pinned.
- **Secondary Stresses:** Particularly in compression members, there are often bending moments introduced due to eccentricity of loads. The loading is eccentric for two reasons, firstly because the timber member is never precisely straight for its whole length, and secondly, the material characteristics mean that the path of stress is unlikely to follow the centre of the member for its entire length. These secondary stresses are difficult to predict.
- **Stress Relaxation:** Studies have shown that timber members experience stress relaxation when subjected to permanent deflections. However, there has been insufficient testing to determine quantitatively just how much stress is lost, and how long it takes for stresses to drop within certain limits. This means that the bending moments in curved flitches, which are typical in Allan and de Burgh trusses, are changing with time and difficult to quantify.
- **Laminated Timber Bottom Chords:** Laminated timber bottom chords are used in both Old PWD and McDonald Trusses. These consist of relatively short lengths of timber bolted together in groups of either three (in Old PWD trusses) or four (in McDonald trusses). The distribution of forces between the various members is impossible to determine accurately, and the effect (if any) of the steel splice plates in sharing either tension or bending moment is also debatable. The distance of bolts from the edge of the timber has a significant effect on both the capacity of the connection and the load distribution between the timber laminates.
- **Thermal Conductivity:** Thermal conductivity is a measure of the rate of heat flow through the material when subjected to a temperature gradient. The thermal conductivity of wood is very small. For example, the thermal conductivity of aluminium is about 1700 times as great, steel 400, concrete 10 and brick 6 times as great as timber. The rate of flow of heat along the grain in timber is about 2.5 times that in the radial and tangential directions.
- **Coefficient of Thermal Expansion:** The coefficient of thermal expansion is a measure of the change in dimension caused by a change in temperature. For wood containing moisture (as is the case for all timber in bridges), though a rise in temperature will tend to make the wood increase in size due to thermal expansion, it will also tend to shrink because of consequent loss of moisture if the temperature rise is maintained for a significant period, and so the net result could well be shrinkage rather than expansion. This means that structural modelling becomes further complicated when other materials are introduced to a timber bridge, such as steel (to strengthen a bottom chord) or concrete (in a timber / concrete deck).

4.1.4 "No Reason to Suspect the Extent of our Ignorance"

Despite the fact that there are a lot of things we do not know about timber, there is one thing that we do know: many beautiful, durable, strong bridges have been built out of it. Significant advances are being made in timber products. The material's natural characteristics (variable strength and elasticity and propensity for moisture movement and long-term creep under load) are being mitigated by better forestry, quality-controlled selection, and engineered wood products produced by a laminating processes and applications of pressure and humidification to set a timber's condition. Genetically modified timbers for structural use are yet to appear.

One of the oldest timber bridges still standing in Europe is the Kapellbrüke in Luzern, built in 1333. Over the centuries much of the structure has been rebuilt, and part of the bridge was destroyed by fire in 1993. However, rebuilding and repair following the original form and carpentry was undertaken and the bridge was reopened in April 1994. The protection of the timber by means of a pitched roof with a large overhang has allowed this bridge to survive.

Figure 73: Photograph taken in 2012 showing the Kapellbrucke in Luzern, built in 1333

In addition to a number of timber bridges in Europe which are many hundreds of years old, there are also a significant number of bridges in North America which are now up to 200 years old. At one time, the United States reportedly had 14,000 covered timber bridges, and many hundreds survive to this day. The era between 1790 and 1860 witnessed the development and patenting of many new bridge designs in the US. Again, they are protected by a pitched roof.

Figure 74: US Covered Timber Bridge

Figure 75: Longest remaining span (210 ft) in US

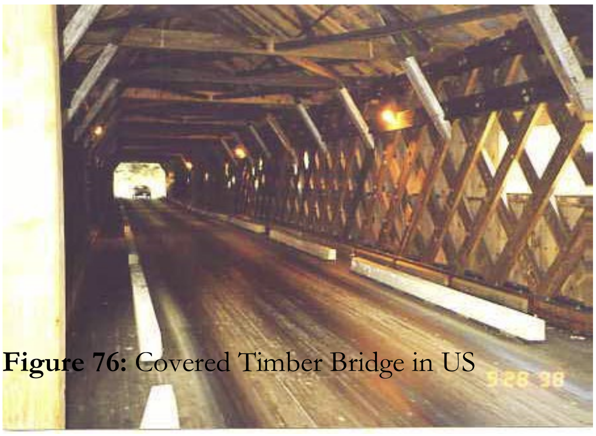


Figure 76: Covered Timber Bridge in US

Figure 77: Another Covered Bridge in US

Closer to home, New Zealand contains a number of timber bridges constructed with Australian hardwoods, that have been standing (uncovered and exposed) for approximately 100 years.

Figure 78: Timber Bridges in NZ using Australian Timber (photos by Lloyd Smith, 2008)

Figure 79: More Timber Bridges in NZ using Australian Timber (photos by Lloyd Smith, 2008)

However, probably the most impressive use of timber for road bridges is the recent work that has been done under the Nordic Timber Bridge Program. An increasingly important aspect of engineering is the consideration of sustainability in design. There are clear environmental benefits associated with the use of timber for bridges. Wood is a natural, renewable material, and wood products typically require less energy to make than alternative materials. Growing trees absorb carbon dioxide from the atmosphere and store the carbon so that about half the dry weight of a tree is carbon. This carbon remains locked up for the life of the wood, even when it is used for construction of timber buildings or bridges. In the 1990s, the Nordic Timber Council took steps to plan the introduction of more timber bridges to the public road network.

Figure 80: Norwegian Timber Bridges (photos from Nordic Innovation, 2002)

Figure 81: More Norwegian Timber Bridges (photos from Nordic Innovation, 2002)

There are now at least 800 modern timber bridges in the Nordic Countries, and although many of these are pedestrian and cycle bridges, some also carry full highway loading. The aim of the Nordic Timber Bridges Project was to improve the competiveness of timber for use in the construction of bridges compared with other materials such as steel and concrete. Many of the bridges make use of relatively modern technologies such as stress laminated timber decks, as well as modern engineered wood products such as glue-lam. There has been enormous research and design effort in detailing these bridges for durability as well as for strength, and the result has been a large number of safe, durable, beautiful, sustainable and economical timber bridges!

4.2 Modelling Timber Beam Bridges

In order to realistically model a timber beam bridge, an engineer must have an understanding of the different elements of the structure how the loads are transmitted from one to another. Due to the fact that exposed timber has a limited durability, the original drawings often do not reflect the actual arrangement on site. Bridges built prior to 1940 did not originally have timber sheeting, and truss bridges designed prior to 1886 did not have kerbs. Details for connecting the various elements together have also changed significantly over time. It is important when modelling an existing bridge or a new design to model the bridge as it actually is (or will be).

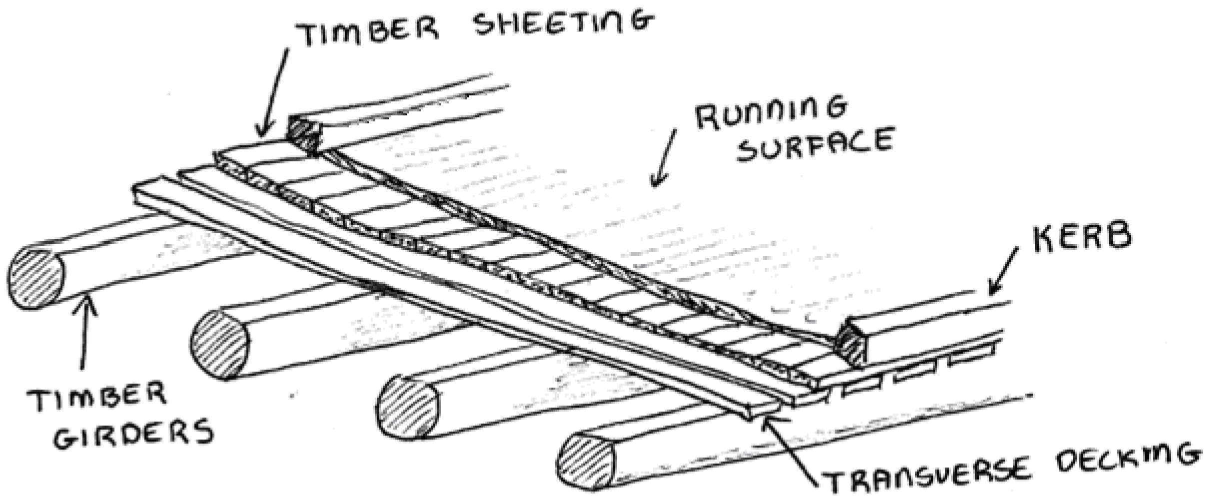


Figure 82: Sketch of Typical Elements of Timber Beam Bridge

The wearing surface generally consists of a spray seal, between 10 and 20 mm thick. This does not contribute to the structural capacity, but is important to include as part of the self weight. Similarly, the kerb does not add to the strength or stiffness of the bridge, but to the self weight.

Under the wearing surface there is generally longitudinal timber sheeting. This may be between 50 and 125 mm thick, and is closely spaced to allow application of the spray seal. Butt joints in longitudinal sheeting are generally staggered, and sheeting is bolted to the transverse decking at each end, and at approximately 1.5m centres along its length. Sheeting does not contribute substantially to the structural capacity of the bridge, but does distribute wheel loads to the transverse decking. It is also possible that thicker longitudinal sheeting (100 - 125 mm) with tight butt joints may transmit longitudinal breaking forces to the abutment by compression.

The transverse decking distributes the loads to the girders, and generally consists of 75 to 125 mm thick elements whose length is equal to the width of the bridge. The transverse decking is generally placed with gaps between elements for durability reasons, and these gaps are generally 20 to 50 mm wide. Each element of transverse decking is generally attached to a bolting strip, which allows construction, but does not contribute to the strength or stiffness of the bridge.

Transverse decking is only irregularly connected to the girders, and so it is difficult for this decking to transmit significant longitudinal or transverse loads to the girders. It is also unrealistic to assume that the girders provide any moment restraint to the transverse decking. Although physically the decking is continuous over the girders, it will rarely act as a continuous member unless all the bolts across the deck (for all the decking) are very tight at all times. This is impractical, rarely achieved and never maintained. While it is possible to analyse a timber girder and decking system as a grillage, this assumes that the deck is a two-way continuous structure and will provide an un-conservative result unless extreme care is taken.

A reasonable approach to this problem has been provided by Kym Wilkinson in her PhD thesis, which suggests that to allow distribution to occur, the transverse members in the grillage may be simulated as the size of three transverse planks at vehicle axle locations only. This relies on the theory that as the vehicle's wheels rest on the sheeting, the sheeting bears onto the transverse planking causing it to deflect, and thereby creating a quasi connection to the girders that then allows distribution. Moment releases must be provided at each end of the transverse planking.

In most cases, the behaviour of timber beam spans can be approximated as simply supported. The effective length of a span is highly dependant upon the corbel arrangements, including the length of corbel, the number and location of girder to corbel connections, and bolt tightness. Due to the fact that all of these are variable, it is appropriate to model the worst case, which assumes that there is no continuity afforded by the corbels, and no negative bending moments.

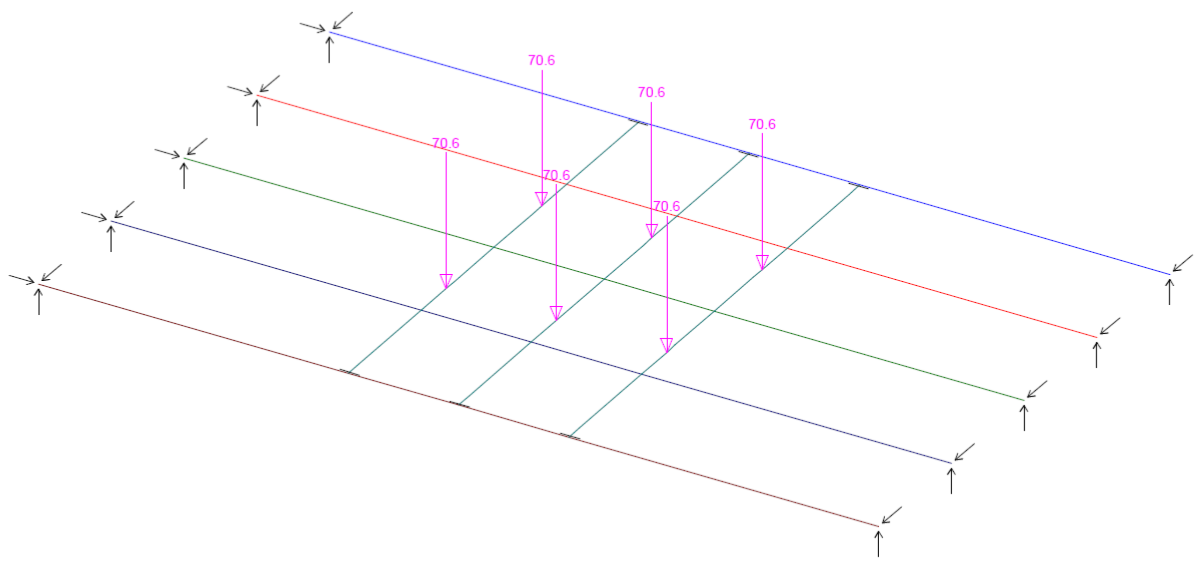


Figure 83: Possible Microstran Grillage Model for Timber Beam Span (loaded with semi trailer tri-axle)

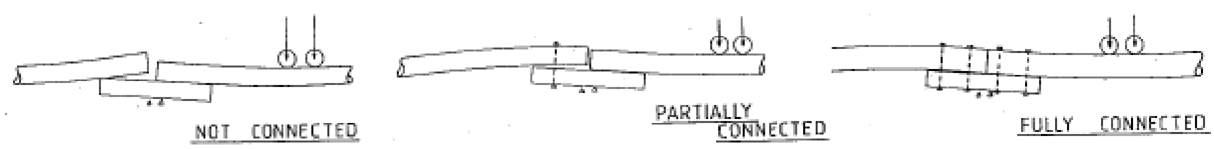


Figure 84: Possible Behaviours of Timber Beam Spans over Corbels (from Yttrup, Law & Audova)

The load taken by each element of transverse decking can be determined as follows:

There are very few bridges which do not have longitudinal sheeting. Generally, these bridges cannot take very heavy loads, and are only found in very low speed environments. For timber decks that do not have any sheeting, only those components directly in contact with the design wheel (tyre) load will share the load. This will depend upon the following variables:

- Design load under consideration (i.e. T44 or W7);
- Orientation of the decking (transverse or diagonal); and
- Width of the decking.

Typically, the decking is transverse and is usually wider than 200 mm. In this case, the wheel contact length of 200 mm will be carried by only one deck plank. The span of the decking without sheeting should be taken as the clear distance between the supports and assumed to be simply supported. Although physically the decking is continuous over the girders, it will rarely act as a continuous member unless all the bolts across the deck for all the decking are very tight at all times. The latter is impractical, rarely achieved and never maintained. The conservatism introduced by assuming a simply supported span is offset slightly by assuming the span is the clear distance between supporting girders.

For most timber decks, the transverse or diagonal planking is overlaid with longitudinal sheeting, and considerable additional distribution of load can be assumed to take place. The number of deck planks sharing the load will depend upon the following variables:

- Design load under consideration (i.e., T44 or W7);
- Orientation of the decking (transverse or diagonal);
- Width and depth of the decking; and
- Depth of the sheeting.

Typically, the sheeting is longitudinal on transverse decking. As a minimum, the load can be assumed to disperse through the sheeting and decking at an angle of 56° (consistent with the principle of disregarding design shear actions within a distance of 1.5 times the depth of a member). The distribution width (in the direction of the traffic) would therefore be equal to (contact length = 200) + (3 x depth of sheeting) + (depth of decking) rounded up to the nearest full number. For example, with 75mm sheeting and 100 mm decking, the assumed distribution width for the deck is 200 + (3 x 75) + 100 = 525 mm. The number of deck planks supporting the load may then be calculated as (distribution width) / (plank width), so for 200 mm wide decking, this gives 2.6 which would then be rounded up to 3 deck planks.

For decks with 100 mm or thicker longitudinal sheeting, the number of transverse deck planks supporting the load may be assumed to be 5, as the stiffness of the sheeting is then equivalent to the stiffness of the planking, causing a greater distribution of load to occur.

The load taken by each timber girder depends upon the following:

- **Number of timber girders:** There are generally four, five or six timber girders, depending upon the deck width and the span length. Clearly, if there are more girders in a span, then each individual girder carries a lower share of the total load.
- **Thickness of transverse decking:** The thicker the transverse decking, the more effective it will be in distributing vehicular live loads to a larger number of timber girders. However, if relatively thin transverse decking is used for a relatively wide deck, then only the timber girders directly under the applied load will share the load.
- **Relative stiffnesses of timber girders:** Stiffness of a girder is dependant upon the size (diameter) as well as the modulus of elasticity (material property). Both of these are subject to significant variability. The stiffer the girder, the more load it will attract. For this reason it is best to minimise the variability in size of a girder in any given span. If one girder needs to be replaced, it is best to replace it with a girder of a similar size to those around it. If a larger girder is provided, it will attract higher loads, and may fail prematurely due to higher loads. Similarly, if there is an undersize girder in a span, it is best to replace it even if it does not show signs of overstress, so that the girders to each side of it do not become overloaded.

A less common form of timber girder span makes use of compound girders. These are generally found in spans exceeding 13m. Before any compound action is assumed in the model, it must be verified that the connections are stiff and strong enough to ensure composite action. In many cases, the shear keys originally specified have long since been lost, and there is no longer any significant composite action either between the two girders or between the girder and the corbel.

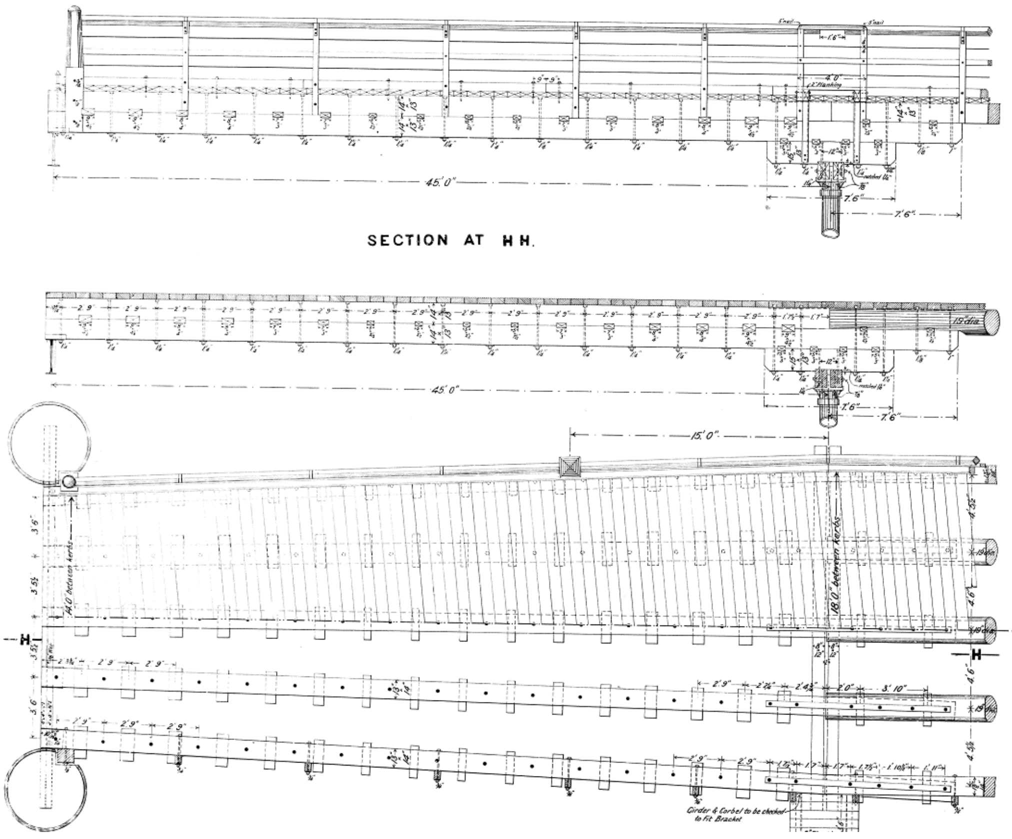


Figure 85: Example of Compound Girder Design for 45' span (McFarlane Bridge, Maclean)

4.3 Modelling Timber Trestle Piers

There are a wide variety of configurations of timber trestle piers used in New South Wales. Some are very short piers whereas others are very tall. Some consist of timber piles driven into the ground and extending up to the bridge superstructure, whereas others consist of timber columns on concrete sill beams or shallow potted piles. There are single trestle piers and double trestle piers, some piers with raked outer piles and others without. There is a variety of bracing arrangements, and some piers have many layers of bracing. Some piers even have horizontal timber sheathing for the total height of the pier. Some piers have a single headstock on top of the timber columns, and others have two headstocks / capwales notched into the sides of the timber columns. For taller piers, there are sometimes splices in the timber, which are not able to accommodate bending moments. Often the site situation is different to the original plans.

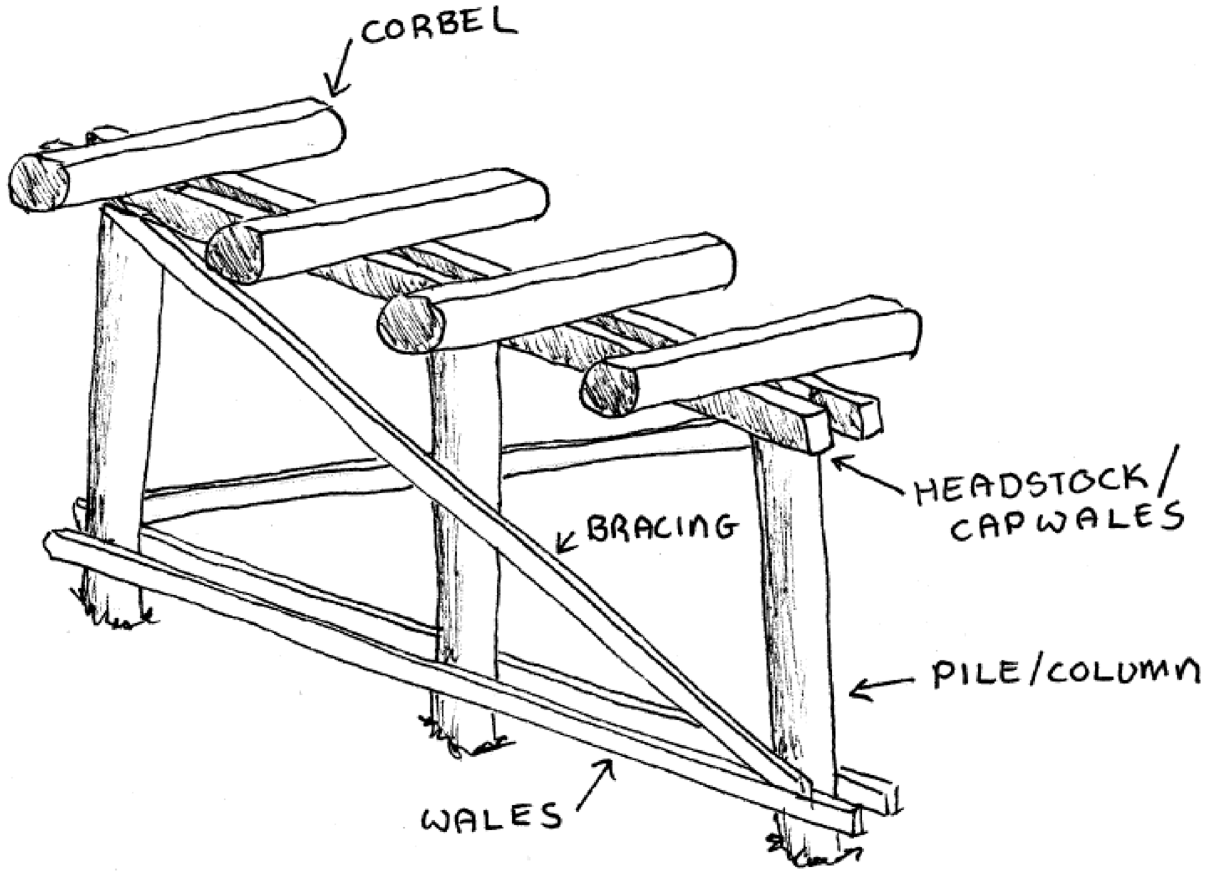


Figure 86: Sketch of Typical Elements of Timber Trestle Pier

It is important to recognise that a full frame model should not be used for timber structures, as such a model does not properly reflect member end conditions and how certain force types can and cannot be transmitted. However, in many cases a full pinned model cannot be used either, or it may miss bending forces that must be considered. Typically, analytical modelling of timber substructures will require an iterative approach to ensure that the force results reflect the actual and practical performance of the structure (and especially the connections) being analysed.

4.3.1 Connections in Timber Trestle Piers

In most cases, butt ended timber members cannot accept tension unless specific connection detailing is applied. In addition, the ends of timber members can rarely be made to resist moments / rotations without significant connections involving steel sleeves or similar. While it is often the case that the timber elements have sufficient capacity to carry all design loads, significant transfer of forces through connections is required. It is often found that the forces experienced in the diagonal bracing under flood and debris loading exceed the capacity of normal bolted connections. Where M24 bolts are specified for loads perpendicular to the grain, the capacity per bolt is less than 15kN, and if too many bolts are used, then the capacity per bolt is reduced significantly, so there is no advantage gained. Laterally loaded bolted connections provide low capacity, and it is recommended that no more than 2 bolts be used per connection.

One of the most critical requirements for detailing timber connections is to avoid connections that attract unwanted loading. Connection details need to retain their strength while not transmitting forces that they cannot handle and subsequently damaging primary components. Too many bolts in a connection will resist too much rotation, and may cause serious splitting.

Another critical factor to remember is that the timber used in timber trestle piers is almost always unseasoned (except in rare cases where engineered wood products such as LVL are used) and so consideration must be given to the effects of shrinkage. 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 the grain. The possibility of restraint to timber shrinkage due to the detailing of bolted joints causes a loss of capacity equivalent to specifying half the number of bolts. In addition to the loss of capacity, there is a risk to durability of the timber through inducing premature splitting and allowing moisture ingress. Joints must therefore be detailed to minimise restraint to timber shrinkage.

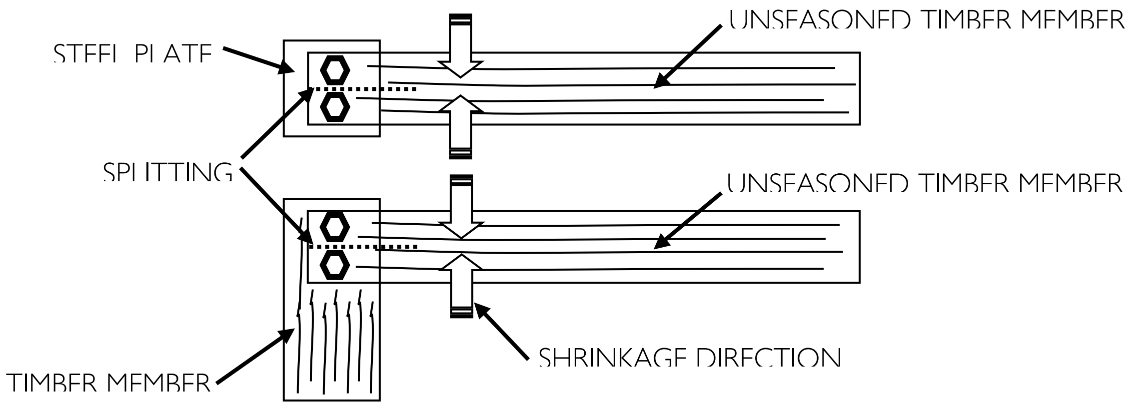
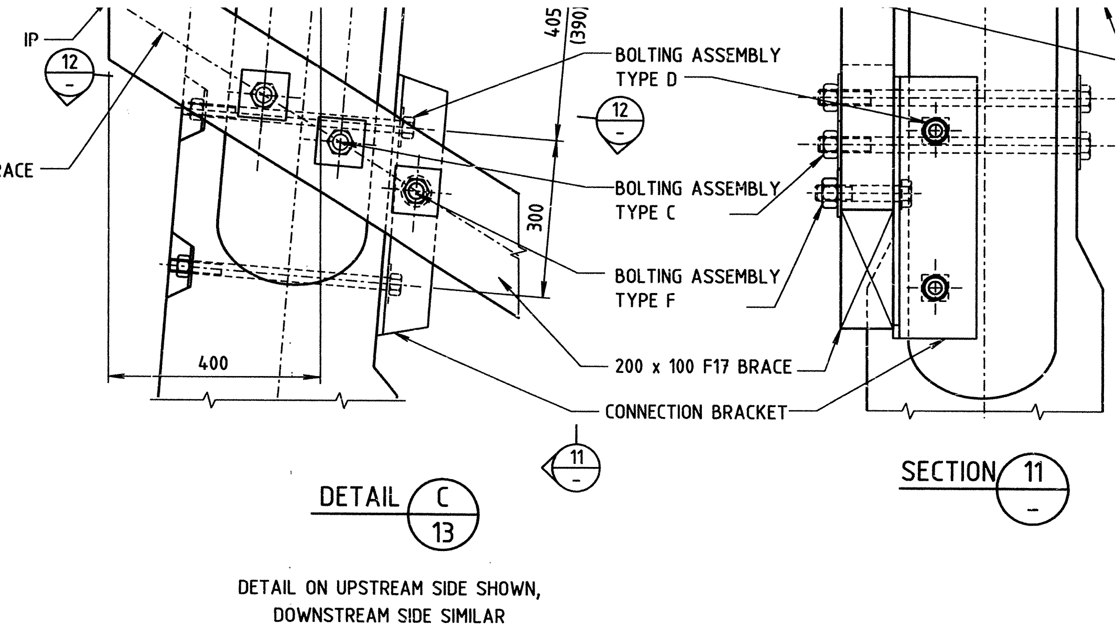


Figure 87: Poor Connection Detailing which Restrains Timber Shrinkage and Causes Splitting

When analysing keyed joints such connections with tang bolts or steel plates with shear keys (found in some splice plates), the designer should not include the capacity of the bolts that hold the plates or tangs in place. This is because the shear resistance of the bolts is too soft compared to the rigidity of the keys, and so the keys would have to fail before the bolts engage.

Similarly, when connections are analysed, the shrinkage of the timber must not be neglected. If a connection becomes ineffective after timber shrinkage has occurred then the connection must not be included in capacity calculations or analysis. This is the case where heavy channel brackets are provided at the bases of piers to attach the columns to concrete footings. It has to be recognised that after the timber has shrunk, the connection is not tight, and so these connections cannot resist bending moments, and must therefore be modelled as pinned connections. Similarly, sometimes in an effort to provide sufficient capacity in a bolted connection of columns to bracing, steel plates or angles are attached in order to allow a greater number of bolts in the connection (see example in figure below). Unfortunately, this fails to take account of timber shrinkage, and it is clear that after shrinkage, the additional bolt cannot carry any load.



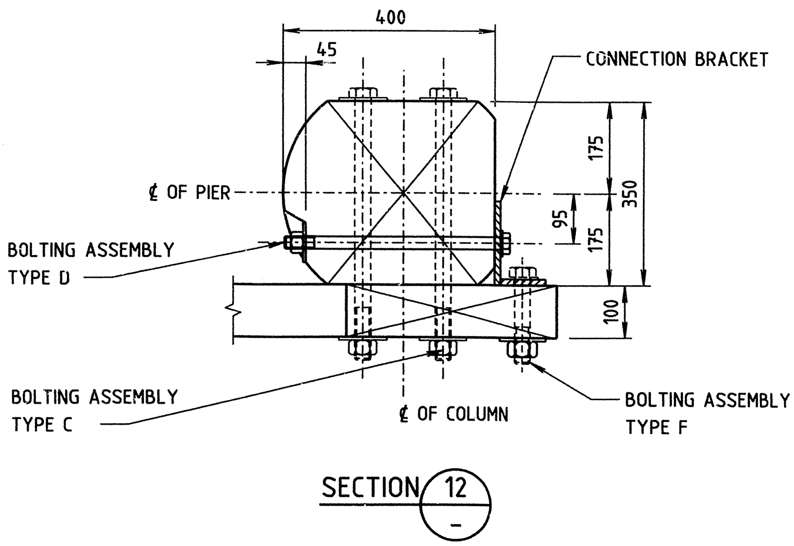


Figure 88: Poor Connection Detailing Neglecting Consideration of Timber Shrinkage

4.3.2 Bending Members in Timber Trestle Piers

For shorter timber piers consisting of timber piles driven into the ground and extending up to the superstructure, the primary load carrying members are the columns, and they act primarily in compression (to take gravity loads) and bending (to take longitudinal braking loads or lateral flood and debris loads). The bracing is less heavily loaded in this type of pier, although it is important in providing lateral stiffness to the pier and in controlling bending moments.

Because timber beam bridges are relatively flexible, the rotation of the beam-corbel system over the pier is significant. As the loaded span rotates, it causes a horizontal movement at the top of the pier away from the loaded span (shown in the figure below). Generally timber beam bridges have piers that are also relatively flexible in the longitudinal direction and so the "pier shoving" does not cause significant distress, but it still must be considered in any pier analysis.

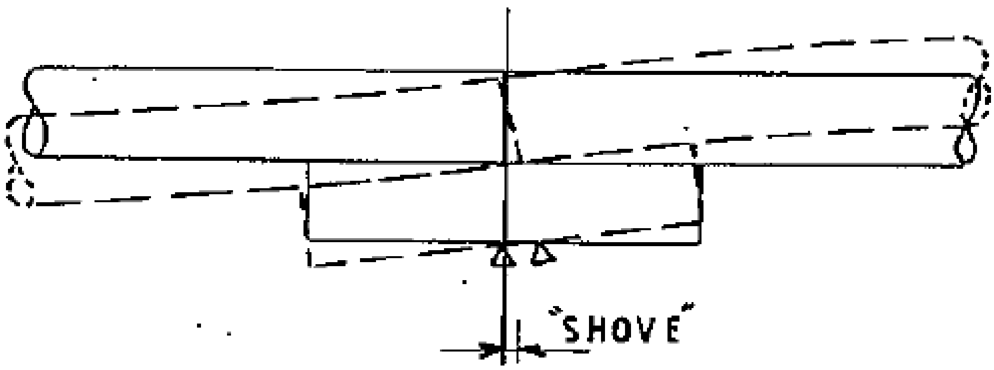


Figure 89: Longitudinal Forces on Piers due to Effects of Corbel Rocking (from Yttrup, Law & Audova)

Another effect of this rotation of the corbel over the pier is that where the corbel is supported by double headstocks rather than a single headstock, it cannot be assumed that both of the headstocks share the load, as it is more likely that the load will be taken only by one headstock. Any computer model must accurately reflect this distribution of load. Furthermore, because the load is only taken by a single headstock, there is eccentricity in the load applied to the columns, so the columns must be analysed taking into account the resulting bending moments.

Trestles with piles continuous into the ground provide some level of restraint to longitudinal forces through the bending capacity of the columns. Taller trestle piers founded on concrete sill beams cannot resist these longitudinal forces and therefore rely on the continuity of the superstructure tying each end to the abutments for their longitudinal stability. Global stability and robustness of the structure under longitudinal and lateral forces must be carefully considered. In any design of new substructures, it must be ensured that stability and robustness are not reduced. This means that it is undesirable to replace large numbers of approach span timber trestle piers with shorter trestles on concrete sill beams. Although this may improve durability, it has reduced global stability and structural robustness, and increases the risk of a "domino effect" type bridge failure. In any new pier design, it is also critical that any new displacement (driven) piles are a sufficient distance from original piles and the locations of old piles must be checked.

4.3.3 *Compression Members in Timber Trestle Piers*

For taller timber piers consisting of spliced timber piles or timber columns seated on concrete sill beams, all members in the pier trestle are primary load carrying members, acting either as compression or tension members. In these piers, the bracing is critical for lateral stability. Columns clearly act in compression to take vertical loads, but are also required to carry tension in the case of flood and debris loading, and so column connections must be designed for both.

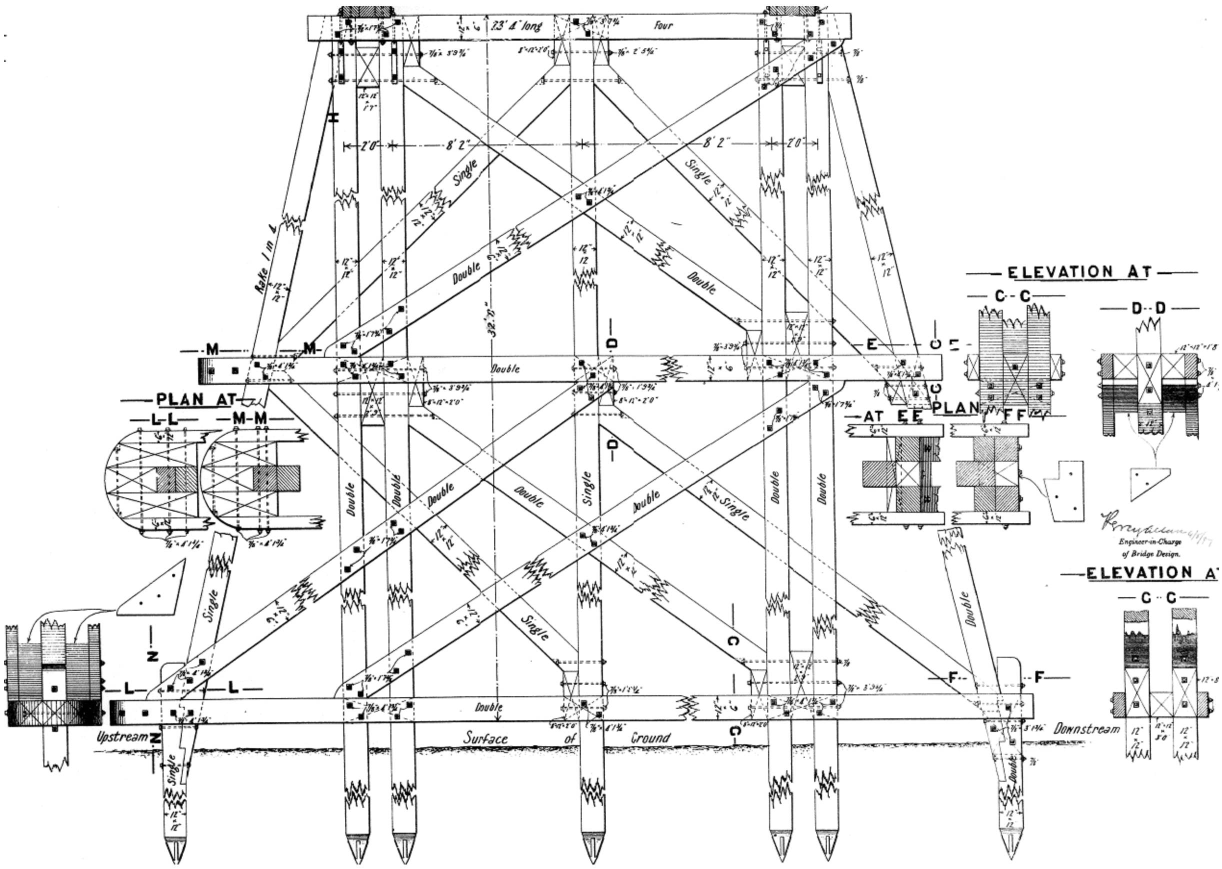


Figure 90: Original Details for Tall Timber Trestle Pier at Vacy Bridge over the Paterson River

In the design above, it is clear which bracing members were designed as compression members, and which were designed as tension members by their connections. The compression members are single members that bear directly against single timber columns, held in place by a single bolt and a timber blocking piece at each end. The tension members are generally double members bolted to the sides of all columns with which they come in contact, with a maximum of two bolts per connection. The arrangement of tension and compression members in the bracing is largely designed to resist flood loading, and it is therefore most effective in the direction of water flow.

The compression members shown above have significantly more capacity than if bolted connections had been used. The controlling factor for the capacity of the connections used above is the bearing capacity perpendicular to grain of the columns and timber blocking pieces. The difficulty with such a connection is the looseness that results from shrinkage of the blocking pieces and the columns. It is critical that regular maintenance is conducted to keep these tight.

4.3.4 Tension Members in Timber Trestle Piers

Tension connections are notoriously difficult in timber. The best tension connections make use of keyed connections and / or bolts in tension. This means that a tang bolt or some variation (such as a keyed plate) is the best option. One helpful property of a tang bolt is that it is external and so there is minimal resistance to member end rotations and so minimal bending moments. Another helpful property of a tang bolt is that it can be readily tightened to adjust for shrinkage. As previously noted, the designer should not include the capacity of the bolts that hold the keys in place because the shear resistance of the bolts is too soft compared to the rigidity of the keys.

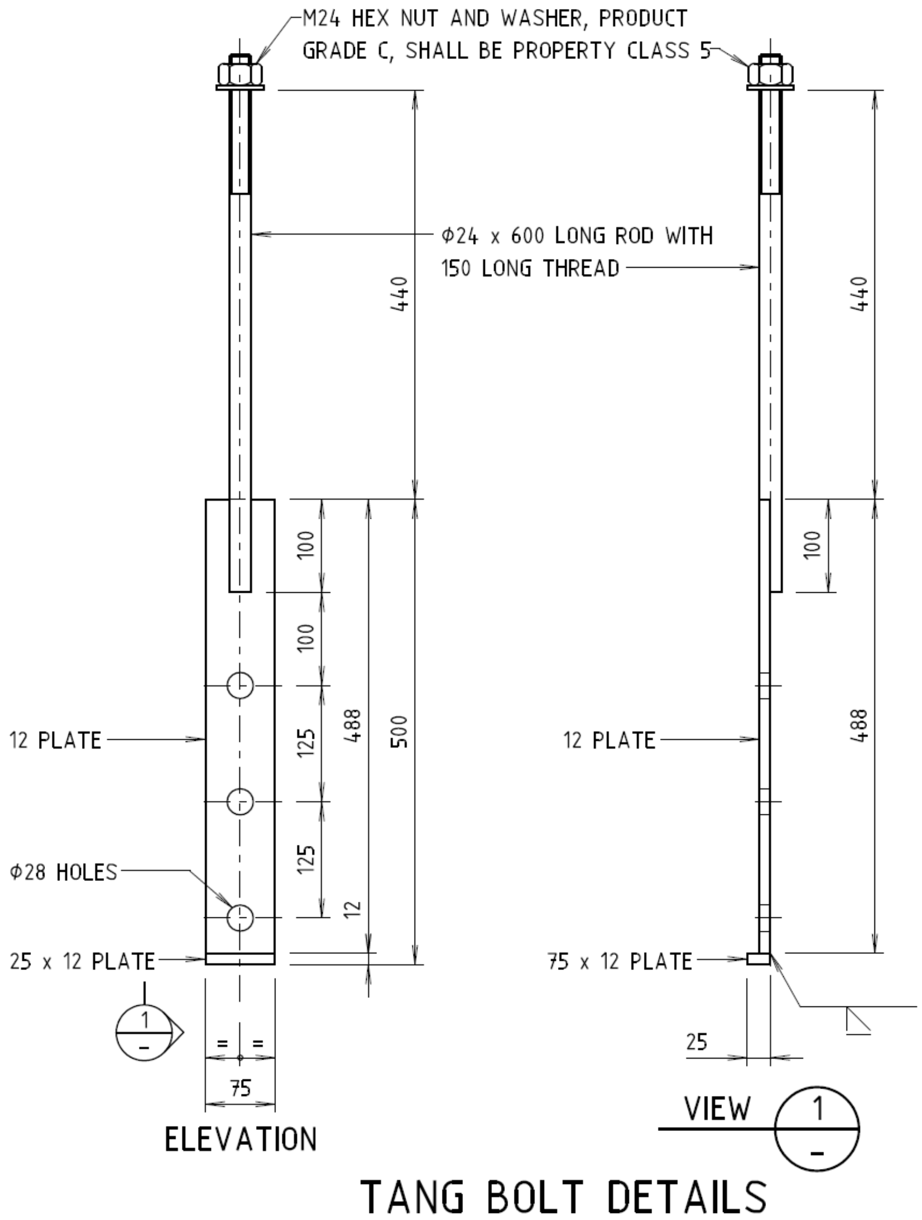


Figure 91: Tang Bolt Details for Tension Connections in Timber

Where tang bolts cannot be used, designers may be able to take advantage of bolts in double shear having twice the capacity of bolts in single shear by having two parallel bracing members.

4.4 Modelling Old PWD Trusses

It is critical when modelling timber truss bridges to take into account the known historical and actual behaviour of these bridges, and especially the known failure mechanisms. This is especially the case with Old PWD trusses, which were the first step in the design evolution of timber truss bridges, and were built before many aspects of timber as a structural material were understood.

Issues associated with Old PWD trusses have been known since the 1880s, and were partly responsible for the later types of trusses being invented. Issues include lack of lateral stiffness of the truss, premature deterioration of the bottom chord and butting blocks, excessive sagging due to elongation of the bottom chord, twisting and warping of trusses, fracturing of cast iron shoes at both ends of principals, and horizontal misalignment of both top and bottom chords.

A very critical component of the Old PWD timber truss bridges is the bottom chord, which consists of three timber laminates bolted together with staggered joints and a single steel plate at every joint. As primary members in timber truss bridges, the bottom chords are subjected primarily to tensile stresses. However, the bottom chords are also subjected to significant bending stresses due to the fact that they support closely spaced cross girders. It is not realistic to model the bottom chord as a solid section because such modelling does not take into account the effects of the discontinuities. It is necessary to accurately model the stiffness in both tension and bending of the bottom chord in order to realistically determine loads and capacities.

The common problem of horizontal misalignment of the bottom chord (even when newly constructed) indicates that loads are not equally distributed between the three flitches, and this needs to be reflected in any analysis. Bolts tend to be very close to the edge of the timber laminates, which has an adverse effect on the capacity of connections. However, the reason for the layout of the bolts, as well as the close spacing of bolts is an attempt to seal the gaps between the laminates in order to stop water from penetrating and causing premature deterioration.

Figure 92: Left: Monkerai Bridge Bottom Chord, Right: Clarence Town Bridge Top Chord

Figure 93: Clarence Town Bridge – Fractured Cast Iron Shoe and Supplementary Tension Rods

Figure 94: Clarence Town Bridge – Sagging of Bottom Chord with Supplementary Under-Trussing

Figure 95: Left: Monkerai Bridge Bottom Chord; Right: Clarence Town Bridge Butting Block

The first step of any bridge modelling is to prepare a global model of the truss. Modelling means the simulation of a physical structure by means of a substitute analytical or numerical construct. It is not simply preparing a framework of nodes and elements. Modelling requires that the physical behaviour of the problem be understood well enough to choose suitable element types (normal, gap, tension-only, compression only etc), section properties, material properties, connections (rigid or pinned) and supports (free to rotate or not, free to translate or not). Results always have to be checked to make sure that they are reasonable. Checking is very important because it is easy to make mistakes in describing the problem to the software. The more detailed the model, the more room there is for mistakes, and the more important it is to check the results.

Depending upon which areas of the truss are of interest, either a two dimensional or a three dimensional model may be used. In an Old PWD type truss, the support conditions are especially critical. Usually the support conditions would be pinned (free to rotate), and one end would be fixed in position while the other end would be free to move horizontally (as shown in Figure 96). Where this is correctly modelled, the tension in the bottom chord should be approximately equivalent to the compression in the top chord, as shown in Figure 97 below.

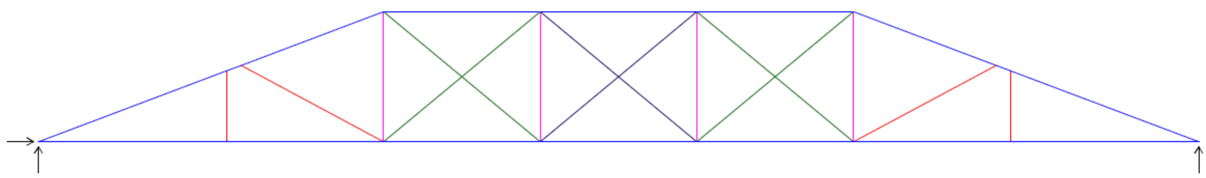


Figure 96: Model Showing Support Conditions for Single Truss Analysis (Old PWD Truss)

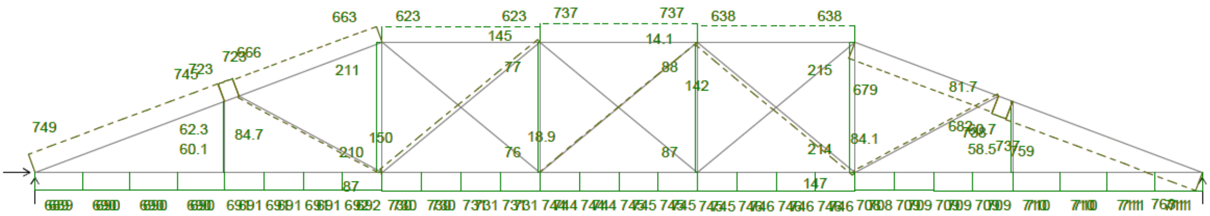


Figure 97: Axial Force Results for Dead Load (top chord compression ≈ bottom chord tension)

If the supports are incorrectly modelled as fixed supports (incapable of horizontal movement), then the tension in the bottom chord becomes almost negligible compared with the compression in the top chord as shown in Figure 98. It is therefore behaving like an arch rather than a truss, which does not reflect the physical reality. This is a critical check if modelling Old PWD trusses.

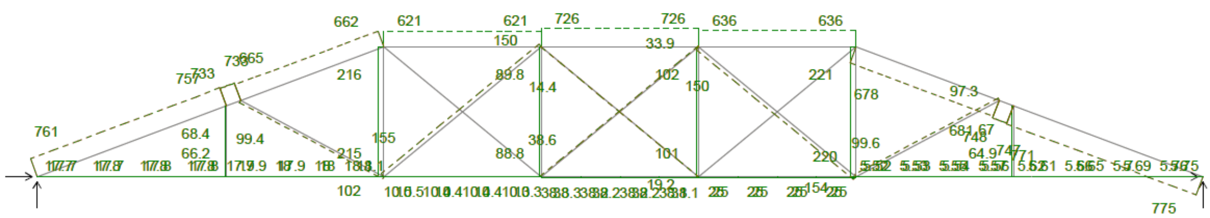


Figure 98: Incorrect Axial Force Results Where Both Supports are Fixed (no horizontal movement)

4.4.1 Modelling of Bottom Chords

A detailed analytical study of the behaviour of the bottom chord in a typical Old PWD truss is carried out here. There are two possible modes of failure for a laminated timber bottom chord. One is fracture of two timber laminates at a joint, and the other is failure of the bolted connections along a shear plane (shown in red in the figure below). The strength can be readily calculated for both failure modes, with the lowest strength governing. Generally, the critical case will be shear of the bolted connections, and in some cases, the code capacity for this failure mode is very low. Here it must be understood that AS 1720.1-2010 does not give an accurate method of calculating the capacity of bolted connections (this currently being worked on by the code committee), and so the designer should not be overly alarmed by the capacity obtained.

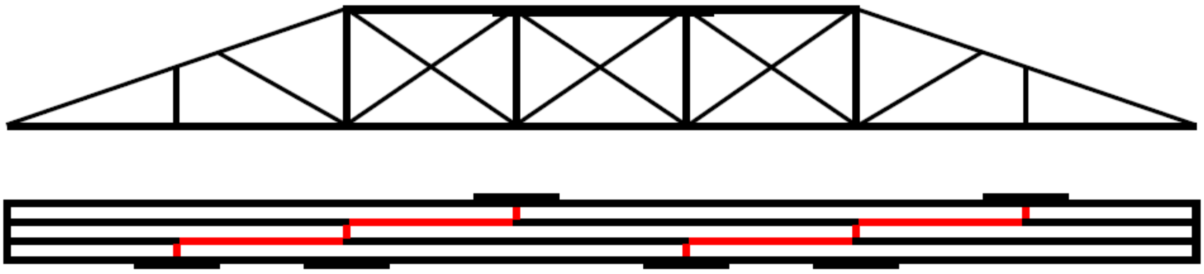


Figure 99: Layout of Laminates and Possible Failure Planes in Old PWD Timber Bottom Chord

To determine the bottom chord stiffness, the three timber laminates of Clarence Town Bridge (as originally designed) are modelled in Microstran (see Figure 100). The centre laminate is 150 mm wide and 355 mm deep, and the outer laminates are 127 mm wide and 355 mm deep.

Laboratory testing carried out in 2011 showed that the stiffness of similar connections (large diameter bolts loaded in shear in large section unseasoned timber) could be modelled as bolts fixed at the centreline of the timber member. In Old PWD trusses, there are generally two sizes of bolts used, with larger bolts located at the splice plates, and smaller bolts in between. At Clarence Town, there are ten 1” bolts at each splice plate, and seven ¾” bolts between each splice plate. However, because four of the ten bolts in the splice plate are less than 150 mm from the edge of the timber, they are not considered effective and are therefore not included in the analysis. The splice plates are not thick enough to provide rotational restraint to the bolts, and so the connections between the bolts and the splice plate must be modelled as pinned connections.

The eccentricity of the splice plate has an important effect, and so the actual location of the splice plate must be modelled (see Figure 100). However, splice plates at internal joints need not be modelled because they make negligible contribution to the strength or stiffness of the member.

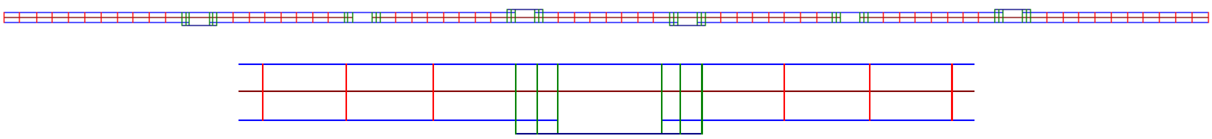


Figure 100: Detailed Model of Old PWD Laminated Timber Bottom Chord (with Splice Detail)

A global truss model is used to determine the tension in the bottom chord, and then that tension is applied to the bottom chord model. The tension induced in the bottom chord of Clarence Town Bridge (as originally constructed) with a T44 truck is approximately 535kN per laminate row, including dead load (Load Factor = 1.4) plus live load (DLA=20%; Load Factor = 2.0).

As can be seen in Figure 101, the bottom chord displays significant horizontal deflection despite the forces being applied purely in tension. According to this model, the out of plane deflection is 65 mm. The bottom chord also displays significant elongation, in this case 35 mm. This is approximately equivalent to the expected elongation if only two of the three laminate rows were active. Alternatively, it is equivalent to reducing the modulus of elasticity of the timber by 45%.

Figure 101: Horizontal Deflection of Old PWD Bottom Chord under Pure Tension Load

The distribution of tension forces between the three rows of laminates is another important aspect to be studied. Figure 102 shows that the tension forces are not equally distributed between the three timber laminates, and the maximum force in a single laminate is approximately 850kN, which is just slightly more than 50% of the tension force in the bottom chord as a whole.

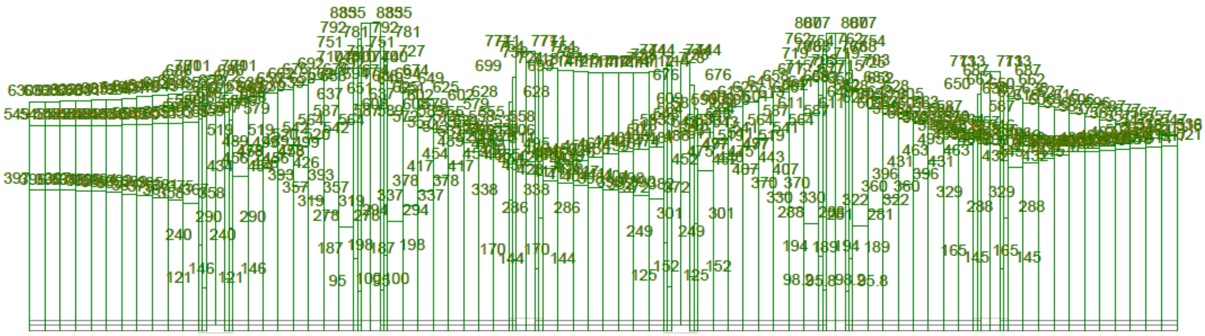


Figure 102: Distribution of Tension Forces Between Three Laminates of Old PWD Bottom Chord

The maximum out of plane bending moment due to tension alone is approximately 7kNm.

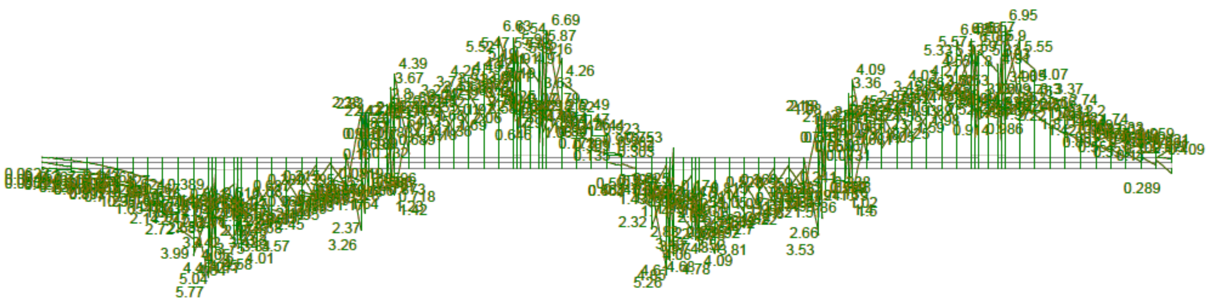


Figure 103: Out of Plane Bending Moments in Timber Laminates of Old PWD Bottom Chord

The next thing that has to be checked in this model is the capacity of the bolts to take the shear loads that have been applied. Because the bottom chord is designed with larger bolts at the splice plates, these attract higher loads. For the example at Clarence Town Bridge, the shear in each bolt is approximately 115kN. Although the steel bolt may have sufficient capacity (if grade 8.8 bolts are used), the code capacity per bolt in unseasoned timber is only approximately 25kN. This means that the bolts cannot effectively transfer the tensile forces into the splice plates.

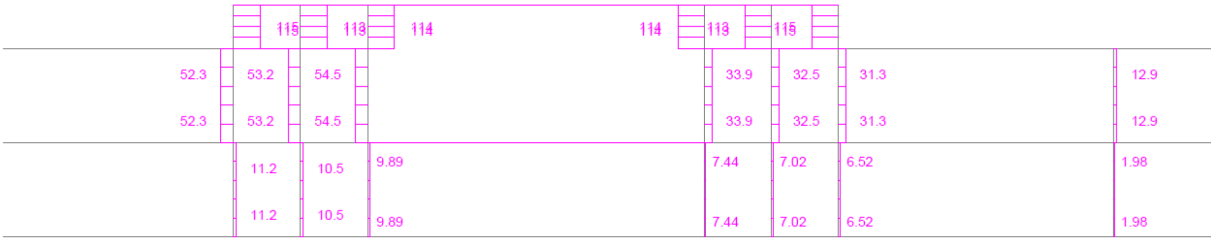


Figure 104: Shear Force in Bolts at Splices of Old PWD Bottom Chord

The bottom chord must therefore be remodelled without the splice plates. When the same force is applied to the new model without splice plates, then the following results:

- Elongation increases from approximately 35 mm to approximately 45 mm.
- Horizontal deflection increases from approximately 65 mm to approximately 200 mm
- The maximum tension force in one laminate increases from 850kN to 1330kN
- The maximum out of plane bending moment increases from 7kNm to 40kNm

It is clear from this analysis that if the splice plates become ineffective, then the bottom chord quickly becomes ineffective with significant out of plane deflection and eccentricity of load paths causing excessive out of plane bending moments and poorly distributed tension forces. All these factors will cause overstressing of components, and accelerate deterioration of the timber.

The strength and stiffness of the bottom chord has a significant impact on other members of the Old PWD trusses. If the equivalent modulus of elasticity of the laminated timber bottom chord is 45% of the modulus of elasticity of timber (due to lack of stiffness in splice connections) then the vertical deflection due to vehicle loads increases by approximately 50%. It is important to note that this additional elongation is not elastic or recoverable because it is a movement at the joints rather than an elongation of the members. This inelastic elongation of the bottom chord means that the truss quickly looses its truss action, thereby causing loss of lateral stability.

It is important to note that these trusses, more than any of the later timber trusses, were designed to behave rigidly. The detailing of the joints and the provision of counterbracing and props for the principal give considerable structural rigidity to the truss. This is advantageous for the most part, but it does mean that the truss is unable to cope with large movements which result from elongation of the bottom chord. When the vertical sag exceeds 50-60 mm, the truss members are unable to make the necessary adjustments, and so the truss tends to lose its ability to perform.

4.4.2 Understanding the Deck: Original, Traditional and Modern

The original deck on the Old PWD truss spans consisted of tightly spaced diagonal decking, 4" (100 mm) thick, attached to cross girders and spiking planks by means of vertical iron spikes, hammered from above. There were no kerbs provided on the truss spans, but kerbs were present on the timber approach spans. Although this decking system was accepted before the introduction of motorised vehicles, by the early 1900s it had become problematic.

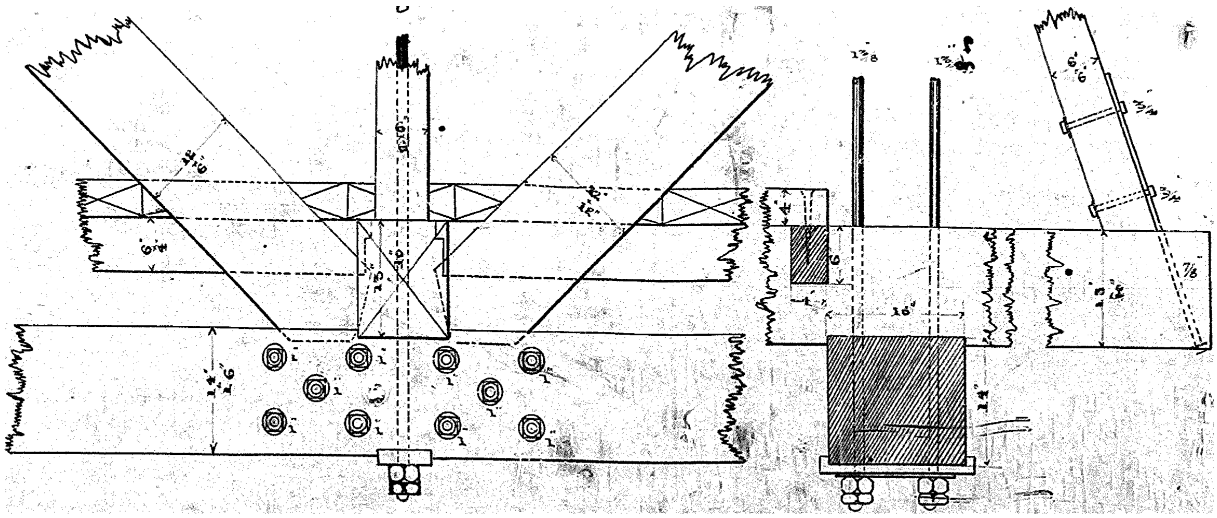


Figure 105: Plan for Original Spiking Planks for Diagonal Decking on Clarence Town Bridge

In 1932 it was reported, "There was a time, not so far distant, when the appearance of a timber bridge was the signal, to the motorist, to slacken speed, on account of the almost universal roughness of timber decks, and to steer clear of projecting spikes. Incautious driving over timber bridges was always likely to take toll of tyres or springs." This problem was certainly experienced at Clarence Town, with records on the files of spikes, "protruding some inches in places above the decking." Exposed timber decks were not a problem only for vehicles, however. Clarence Town Bridge was commonly crossed by teams of bullocks or horses hauling timber or farm produce from when it was first constructed and into the mid-1900s. Timber decks can be slippery, and this was dangerous both for vehicles and for animals. In 1935 a valuable leader of a bullock team had to be put down after it broke a leg by slipping on the timber deck of Clarence Town Bridge, and this led to numerous requests to "tar and sand" the deck or to put gravel on it.

By the 1930s it had become common practice to surface a timber deck with 3" (75 mm) of pre-mix macadam (an early version of asphalt, consisting of layers of compacted broken stone). However, this was generally not applied to timber truss spans due to the excessive extra weight.

By the early 1960s, action had been proceeding throughout the State to attach longitudinal sheeting (generally 50 mm thick) to decks of timber bridges on Main Roads. The traditional construction had become increasingly unsatisfactory under motor traffic, because the planks loosened under the driving wheels, and the consequent uneven riding surface in turn shook the entire structure of the bridge, loosening other members and causing noise when vehicles crossed.

In 1962 it was reported, "By placing longitudinal sheeting above the transverse planks a surface is provided which is smooth and not affected by the tractive effort of the wheels… A possible disadvantage of the application of longitudinal sheeting over existing transverse planks is the likelihood of accelerated decay of the planks… To prevent the development of slippery conditions in wet or frosty weather, a bituminous surface, either flush seal or premix, is applied to the longitudinal sheeting." It was not long before the "possible disadvantage" was realised, and the decking underneath decayed with rapidity. To improve the durability of the bottom layer of decking, gaps were introduced in the 1980s (approx 30-50 mm) between decking planks (whether they be diagonal or transverse) to allow improved drainage and ventilation.

Figure 106: Photograph of Monkerai Old PWD trusses with Original Decking System

Figure 106 shows the original decking system used on Old PWD trusses. Things to observe include the change in timber rail system from the approaches to the trusses (no diagonal top rail and no posts on the truss spans), the lack of kerbs, and the lack of gaps between the diagonal decking planks. There is no wearing surface added, but exposed timber. Note also the lack of bolt heads protruding above deck level, as iron spikes were hammered flush with the surface.

When longitudinal sheeting was introduced, spiking was no longer suitable, but the longitudinal sheeting was bolted to the decking. For this reason, the spiking plank was done away with as no longer necessary, and kerbs were used to support the ends of the decking from above with bolts.

The technical reason for diagonal decking in the original design had been to provide lateral stiffness to the deck. The system of tightly packed diagonal decking spiked to closely spaced cross girders held in place with spiking planks did give significant lateral stiffness, sufficient to resist wind loads and keep the bottom chord in straight alignment. When Percy Allan introduced transverse decking in 1893, he needed to provide under-deck wind bracing for longer spans, and for shorter spans, he relied upon staggered longitudinal stringers bolted to the cross girders.

Now that the spiking planks have been done away with, and gaps have been introduced between the diagonal decking, the lateral stiffness of the system is somewhat reduced (although longitudinal sheeting does assist). Furthermore, with modern tyres, larger horizontal forces are transmitted into the deck due to the increased traction available. The culmination of these issues can be seen at both Clarence Town and Monkerai, where the bottom chords of the truss spans are not following a straight alignment. This indicates that the present decking system has insufficient lateral stiffness to keep the spans properly aligned under current loading conditions.

The history of decks on Old PWD trusses shows the changing use from bullock drawn wagons to light motor vehicles and then to heavier motor vehicles. It also shows the changing community expectations from the early 1900s when vehicles slowed down on timber bridges to avoid damage to their cars, to today when the travelling community expects bridges to be safe.

The next step in the evolution of decks on timber bridges has been the stress laminated timber (SLT) deck. Invented in Canada, this technology was introduced to Australia in the early 1990s, and has been widely applied to timber truss bridges, especially Allan, de Burgh and Dare trusses.

The primary advantage of a stress laminated timber deck applied to Old PWD and McDonald trusses is the greatly improved lateral stiffness. Although the original diagonal decking provided a level of stiffness appropriate for traffic in the late 1800s, the traditional decking and sheeting used since the 1960s does not provide sufficient lateral stiffness for today's heavy vehicles. An SLT deck therefore provides the same function as the original diagonal decking scheme, but with greater effectiveness to sustain today's loads. An SLT deck should be the preferred option when rehabilitating Old PWD and McDonald trusses. Other advantages of the SLT deck include:

- No trip hazards for pedestrians and no groove hazards for cyclists (thereby increasing community safety and reducing litigation risks).
- Improved community benefit (reducing the need for inconvenient bridge closures required approximately every seven years to replace sheeting and every 14 years to replace decking).
- Improved durability for primary members (an SLT deck provides important waterproofing, which extends the life of timber and steel members (such as cross girders) beneath.
- Improved ability to distribute both vertical and horizontal loads (thereby allowing greater flexibility to preserve original member sizes).
- Negligible change in dead load of the bridge (thereby reducing the need to strengthen other parts of the truss in order to sustain dead loads).
- Improved riding surface (giving a more comfortable journey for the community, a quieter bridge for local residents, and also reducing negative effects of vibrations).

4.4.3 Understanding the Approach Spans

One critical aspect of Bennett's original designs was the provision of an alternative load path for the truss to reduce the risk of excessive elongation of the bottom chords leading to excessive sagging. He did this by detailing the approach spans to enable them to provide horizontal support to the truss. For all of the larger truss spans (70' and over), the outer approach span timber girders were aligned with and bearing against the timber butting blocks and are carefully detailed with scarf joints and corbels with shear keys to take any horizontal forces all the way back to the abutments. For this reason, the Old PWD type trusses were able to take vehicular loads significantly greater than those for which they were designed (less than eight tonnes), with an average life exceeding 50 years, and many bridges remaining in service beyond 80 years.

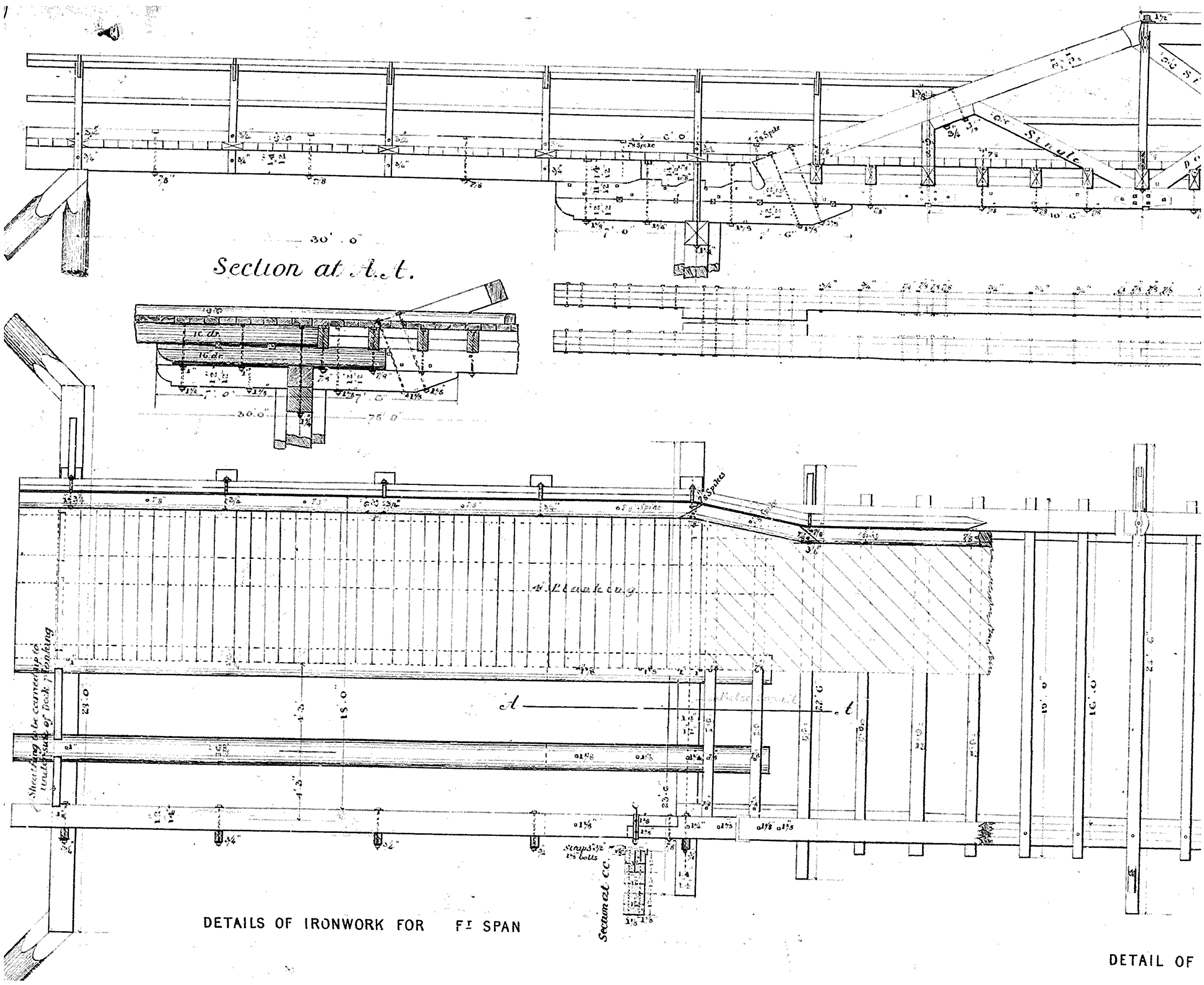


Figure 107: Continuity from principal to butting block to approach span girder to abutment.

Unfortunately, Bennett detailed very long timber girder approach spans (generally 12 to 14m in length) and these approach spans are unable to carry today's vehicular loads, which means that they are often replaced with modern materials such as steel or concrete. Due to the thermal properties of these modern materials, they are unable to provide the same function. If approach spans are modified, it is important that the truss spans are strengthened to make up for the loss.

4.4.4 Common Defects in Cast Iron Shoes

Another critical issue with Old PWD trusses is the detailing of the cast iron shoe at the intersection of the principal with the top chord. These cast iron shoes are generally fitted with wrought-iron washer plates, through which the suspension rods pass. As the timber in the top chord shrinks from under them, these plates become bent, in some cases breaking the cast-iron shoes (Figure 93), and in other cases breaking the wrought iron plate. If these do not break, the surface of the washer plate, being no longer level, nor at right angles to the suspension bolt, has a tendency to bend the suspension rod under the nut thereby breaking the suspension rod.

This effect is further exacerbated by the fact that the centrelines of the various elements being connected at this point to do not neatly intersect at the connection, causing eccentricity in the load paths. The location of the tension rod away from the shoe causes bending in the top chord. It is important that these eccentricities are accurately reflected in any analytical model.

The cast iron shoes at the base of the principals are less critical in that a failure of these shoes does not tend to cause a failure of the truss. It is very common for these shoes to be found fractured. This may be due to uneven bearing of the principal on the shoe. The base of the sloping principal does have a tendency to collect some moisture which accelerates rot, leading to loss of section at the end of the timber, and uneven bearing on the brittle cast iron shoe.

Figure 108: Fractured Cast Iron Shoe at Base of Principal on Monkerai Bridge (2012)

4.4.5 Modelling Connections

The earlier timber truss bridges were detailed to maximise rigidity, and none more so than the Old PWD type truss. The counterbracing of all the panels, and the use of timber props for the principals all restrict the flexibility of the truss. In addition to this, where the counterbracing crosses bracing, notched connections are used in addition to bolts, thereby further restricting relative movements. Even the detailing of the timber sway bracing allowed for very little movement between members. This made the Old PWD type truss very robust, especially under relatively light loads. However, this rigidity introduces problems when movements do occur, either due to extension of the bottom chord, shrinkage of the timbers, or modern heavy loads.

William Christopher Bennett kept an album of photographs of the roads and bridges of New South Wales, which has ended up in Cambridge University Library in England. This album contains photographs of a number of Old PWD type and earlier timber truss bridges. From studying these very early photographs, it seems that the bridges were originally built with little to no camber. However, some later photographs of Old PWD type timber truss bridges seem to show very significant camber, well in excess of 100mm. It seems likely that, as the timber of the top and bottom chords and cross girders shrank, in order to take up the slack, the tension rods were tightened to lift up the bottom chord and so the camber increased with time. The inconsistent geometry of the truss and the rigidity of the connections, however, would mean that some gaps could not be closed by this method, which is why McDonald introduced iron wedges.

Including the connections of the bracing to the counterbracing in a Microstran model generally causes the analysis to fail. On the remaining Old PWD trusses, the rigidity of the connections has often been reduced by over-sizing the notches in order to allow some relative movements. This is especially important if consideration is being given to introducing steel in the bottom chord, which will be subject to thermal movements not originally intended for this truss type. Clearly, if connections are oversize, rigidity is reduced, which may be a structural disadvantage.

Figure 109: Oversize notched joints on Monkerai Bridge to allow Tightening and Recambering

4.4.6 *Understanding the Sway Bracing*

The timber sway bracing as Bennett designed it had tension / compression connections at both ends which were achieved by an arrangement of notches in the timber (to transfer compression) and tang bolts (for tension). This arrangement gave substantial lateral restraint to the top chord. The Old PWD is the only truss type which originally relied on sway bracing for lateral stability.

In the later truss types, the timber sway braces were replaced with iron or steel sway braces, and the number of sway braces was reduced, which was appropriate for the more flexible later truss types. Unfortunately, this practice of providing slender steel sway bracing (angles, channels or T-sections) with simple bolted connections has more recently been incorporated into Old PWD trusses, and has generally been ineffective in preventing loss of lateral stability of this truss type.

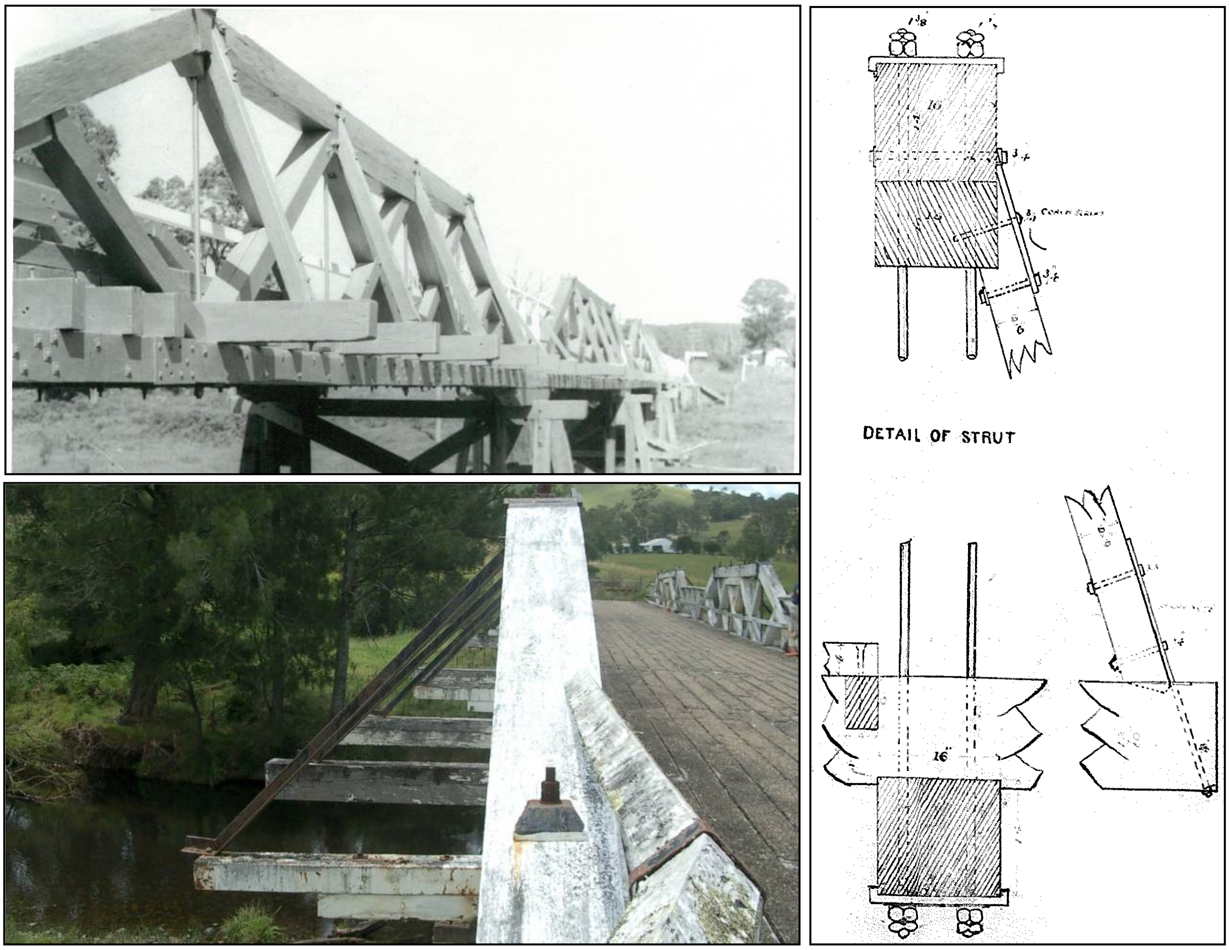


Figure 110: Change from Original Timber to Steel Sway Bracing on Monkerai Bridge

Although the steel sections for sway braces are slender, they do not tend to fail by buckling, but they fail at the connections. The bolts through the top chord (loaded in shear perpendicular to the grain) and through the timber cross girders (loaded in compression perpendicular to the grain) simply have insufficient capacity to restrain the lateral movement of the top chord once the truss begins to sag. The bolts in the top chord have a tendency to split the timber, accelerating rot in those areas, and the bolts in the cross girders simply become loose as gaps open up.

4.4.7 Effects of Using Shorter or Smaller Timber

Bennett designed this truss type in a time when timber resources were plentiful and cheap. He therefore used a truss geometry which depends upon very large sections and long lengths of timber to work effectively. The bottom chord depends, for its strength and stiffness, on very long lengths of timber (up to 18 m for the 100' spans – Fig 112). The top chord and principals depend, for their strength and stability, on very large sections of timber (355 mm x 405 mm for the 100' spans) which are also relatively long (approximately 13 m in length for the 100' spans).

Unfortunately, timber resources are no longer plentiful or cheap, and so modifications have been made to these trusses attempting to make use of shorter lengths and smaller section sizes.

The use of shorter lengths in the laminated timber bottom chord has a detrimental effect on the strength and stiffness of this member for two reasons. Firstly, an increase in the number of connections decreases the lap length for each connection and therefore decreases the overall strength and stiffness. Secondly, shortening the lengths of the timber means that splice connections must be introduced away from the panel points, where they are subjected to different load effects and bolting configurations and are therefore more readily overstressed.

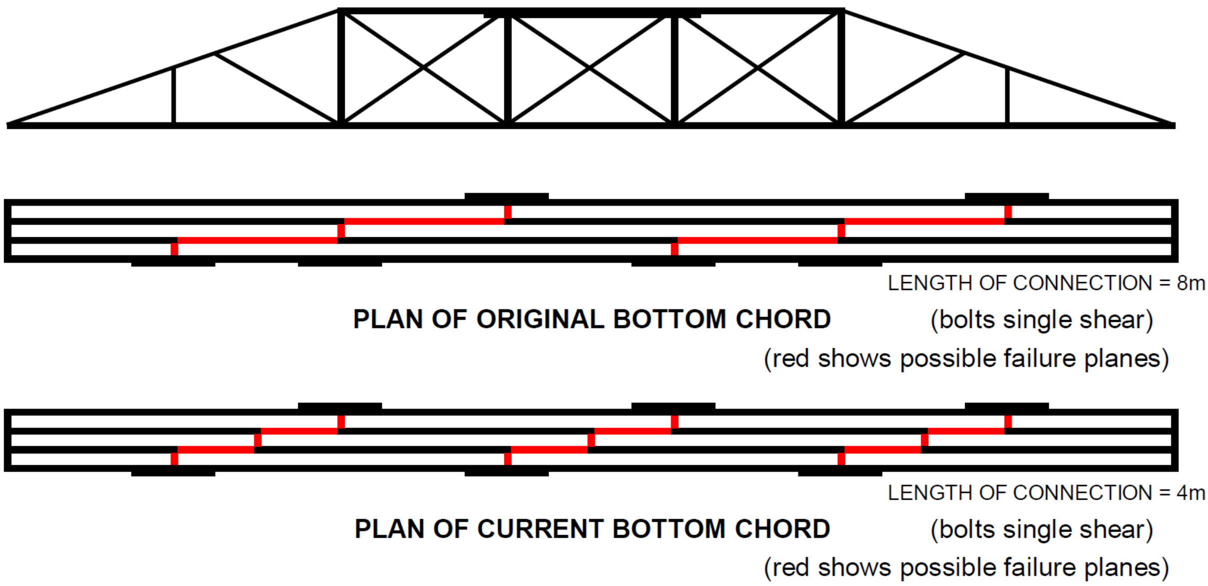


Figure 111: Effects of Reduction in Length of Bottom Chord Timber Laminates by One Third

By reducing the length of the laminates in the bottom chord by just one third (from approximately 12.3m to 8.2m in a 100' span), the effect on the bottom chord is a reduction in strength of approximately 50% and a reduction in longitudinal stiffness of approximately 65%. In addition to this, the closer spacing of the connections means that the eccentric load effects through the bottom chord are accentuated, causing increased horizontal movements and biaxial bending stresses. All these effects, in turn, lead to excessive sagging of the bottom chord, loss of truss action, and loss of lateral stability of the top chord and principals as is seen in Figure 92.

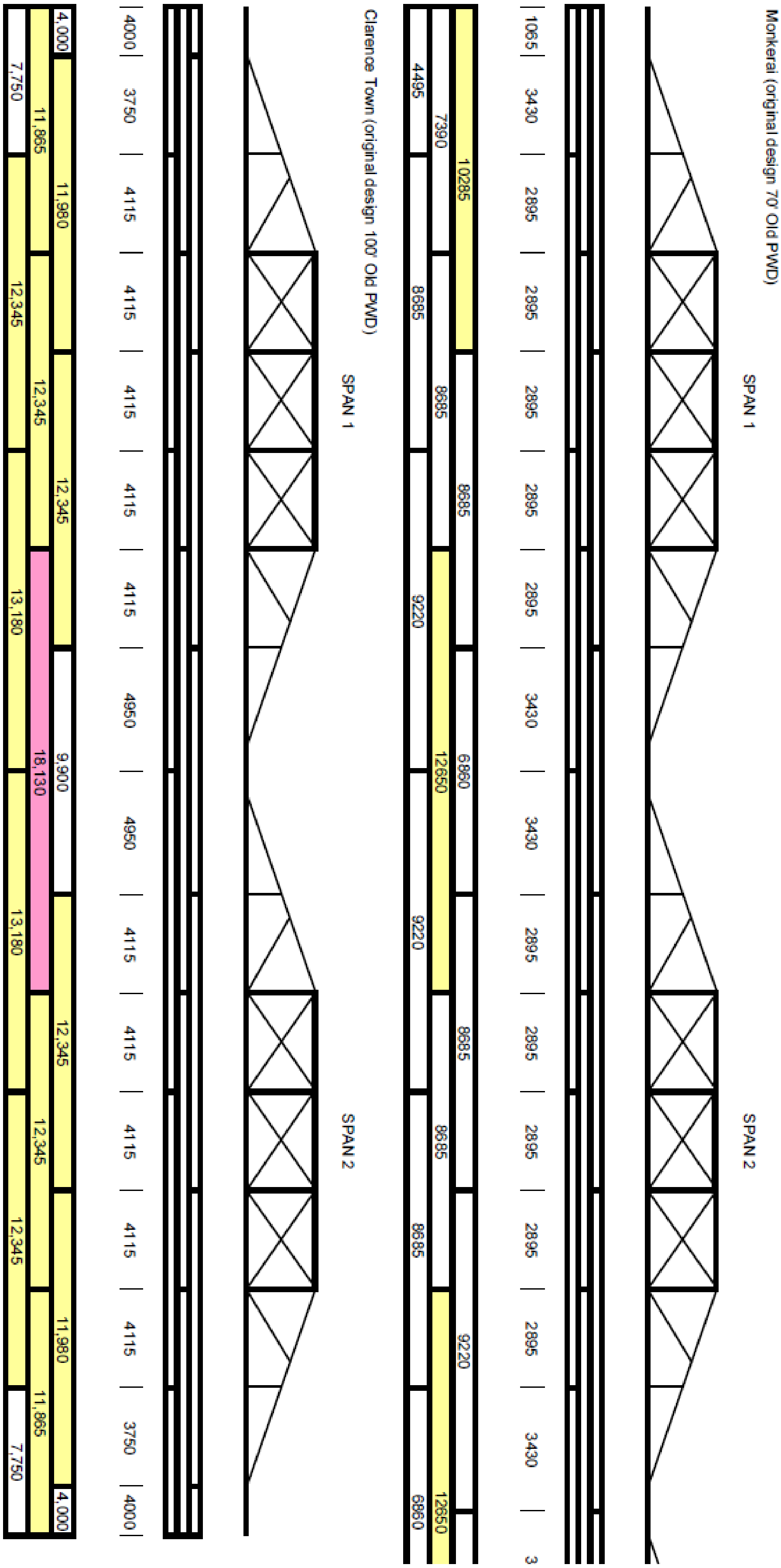


Figure 112: Original Laminate Layouts with Lengths in Old PWD Bottom Chords

Unfortunately, strengthening the splice plates by introducing shear keys similar to those used in Allan trusses does not solve this problem because the layout of the splice plates means that strengthening the splice plates also leads to an increase in eccentric loading, thereby causing additional biaxial bending in the bottom chord. In order to effectively transfer the load, strengthened splice plates must be provided on both sides of the individual laminates being joined, including both external and internal laminates, which would be difficult to construct.

Again, due to the difficulties in obtaining long large sections of timber, some top chords have been replaced with two smaller members bolted together. Despite the large number of bolts generally used to connect the two pieces together, the bolted connection is significantly less stiff than a single member. Not only does this reduce the compression capacity of the top chord element, but it significantly reduces the lateral stability of the truss due to uneven bearing at the top chord cast iron shoes. Again, this has a tendency to cause loss of lateral stability of the truss.

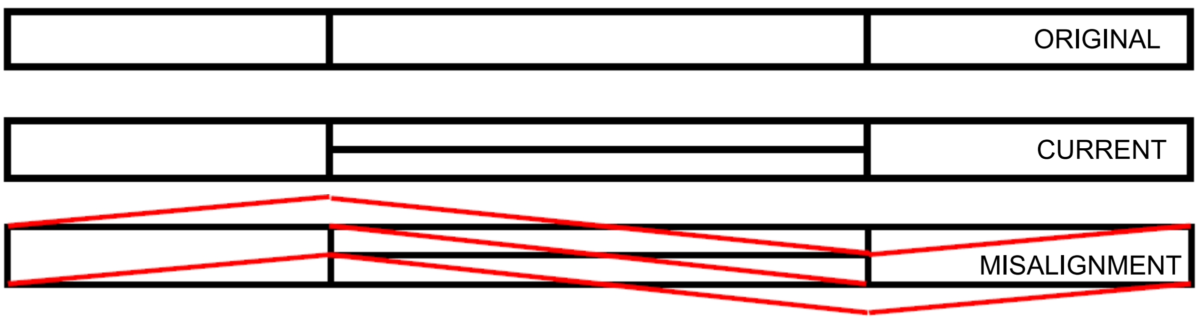


Figure 113: Effects of Replacing Single Solid Top Chord with Double Top Chord Member

4.4.8 Summary

Old PWD timber truss bridges, more than any other timber truss bridge types, are particularly susceptible to second order effects. This means that although they may behave a certain way under small deflections, when deflections reach a critical limit, the behaviour of the truss changes dramatically. For this reason, it is important to analyse the structure using a non-linear second order analysis which takes into account not just the compression-only and tension-only members, but also the three dimensional effects of the truss under various load combination deflections.

In order to obtain accurate results, great care must be taken to understand connection behaviour, so that the model does not overestimate or underestimate the connection strength or stiffness.

When materials other than timber are added to the bridge, whether they be on the approach spans or on the truss spans, then further analysis is required to ensure the compatibility of the new materials with the truss under various different load combinations (including temperature effects). For example, replacement of approach spans with steel changes the behaviour of the truss, as was seen in Section 3.4.2, and any replacement of truss members (such as bottom chords) with steel will introduce thermal movements which must be combined with other load cases to ensure that deflections do not reach the critical limit leading to loss of truss action.

4.5 Modelling McDonald Trusses

According to the 1962 Bridge Maintenance Manual, the camber in a McDonald truss should be approximately 50 mm for a 90' span, 45 mm for a 75' span and 38 mm for a 65' span when first erected, and the chords should never be allowed to approach a straight line. According to the current M757 maintenance specification, a camber of 20 mm should be maintained for all McDonald trusses, noting that McDonald and Old PWD trusses are complex to adjust. According to the original drawings and early photos, the camber in McDonald trusses appears to be zero. Although McDonald introduced splayed principles to increase lateral stiffness, steel wedges for taking up slackness in the braces, and a stiffer laminated timber bottom chord, these trusses still have a tendency to sag and buckle out due to elongation of the bottom chord.

Figure 114: Loss of Lateral Stability in sagging McDonald Truss over Bega River at Tarraganda

Another similarity between the McDonald and the Old PWD trusses is the continuous laminated timber bottom chord with very long lengths. The longest length used in a McDonald truss is the 16.3m length used at the ends of the 90' spans. However, even for the 75' spans, most of the timbers required exceed 10m in length and must be the best quality heart free sap free timber.

McDonald truss bridges are also generally supplied with timber girder approach spans (if any), and the outer timber girder is enclosed within the timber butting blocks, providing a certain amount of additional horizontal propping of the truss back to the abutments. Therefore, many of the modelling considerations that apply to Old PWD trusses also apply to McDonald trusses.

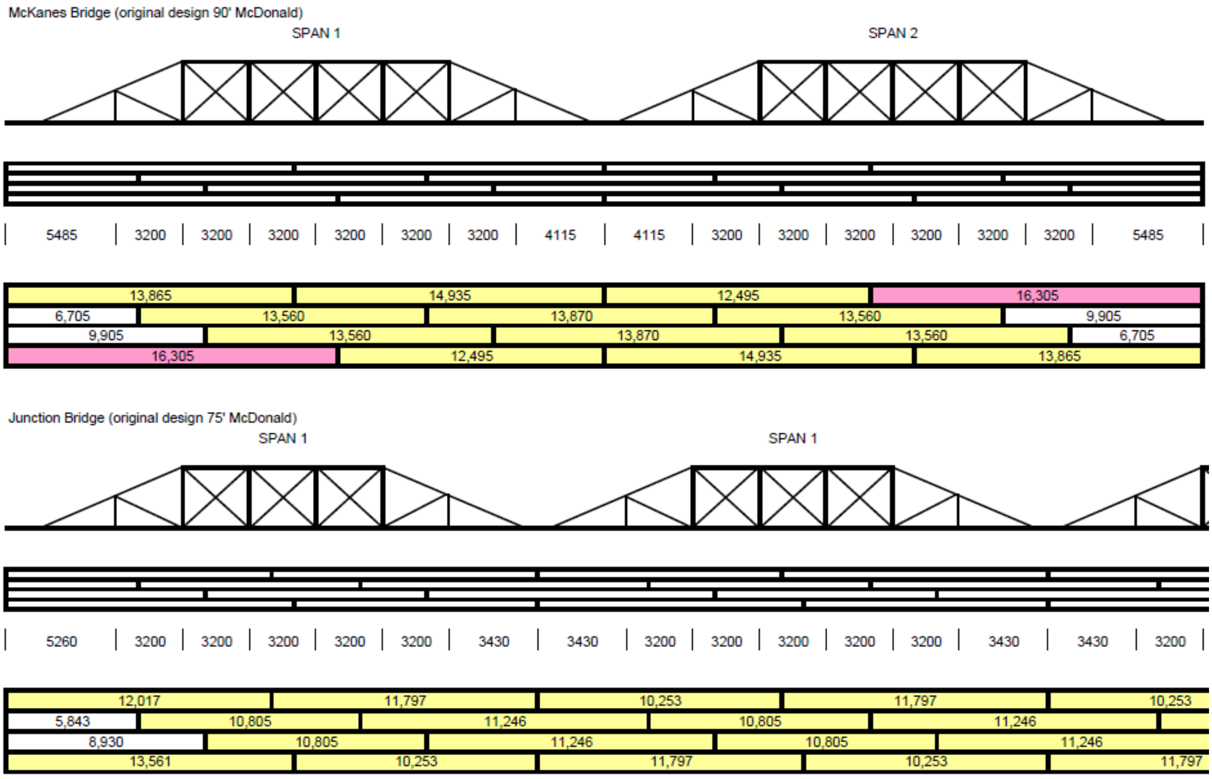


Figure 115: Layout of Laminated Timber Bottom Chord in McDonald Trusses

However, there are aspects that are unique to McDonald trusses and these will be explored here.

4.5.1 Modelling of Principals

While Bennett in his Old PWD truss had taken care to design the timber sway bracing to resist lateral buckling of the top chord, later designers did not take this approach. McDonald, Allan, de Burgh and Dare designed top chords to have sufficient capacity to resist buckling even without sway bracing, and supplied only very slender, low capacity sway bracing to limit vibrations.

In the McDonald trusses, the lateral stability of the top chord is entirely dependent upon the splaying of the principals. In later truss types, the top chords were designed as columns with varying loads, unsupported in the lateral direction. However, in the McDonald truss, the compressive stress in the top chord does not vary so significantly along its length, and it was designed as a column pinned at each end, and unsupported in the lateral direction.

In practice, the splayed principals do not have sufficient strength or stiffness to prevent the top chord from lateral movement, and so these trusses are also subject to horizontal misalignment. There are two primary reasons for this. Firstly, the bases of the principals are bearing against timber, and stresses are acting both perpendicular and parallel to the grain. The base of the sloping principal has a tendency to collect moisture which accelerates rot and softens the timber. Therefore, the butting block and bottom chord on which the principal rests behave more like a spring foundation than a rigid foundation, thereby allowing rotation of the principal at the base.

Secondly, although the timber spacers in the principals of the McDonald trusses are clearly designed to ensure composite action between the two flitches (unique to McDonald trusses), as soon as the timber spacer splits, composite action is lost, and lateral stiffness is greatly reduced.

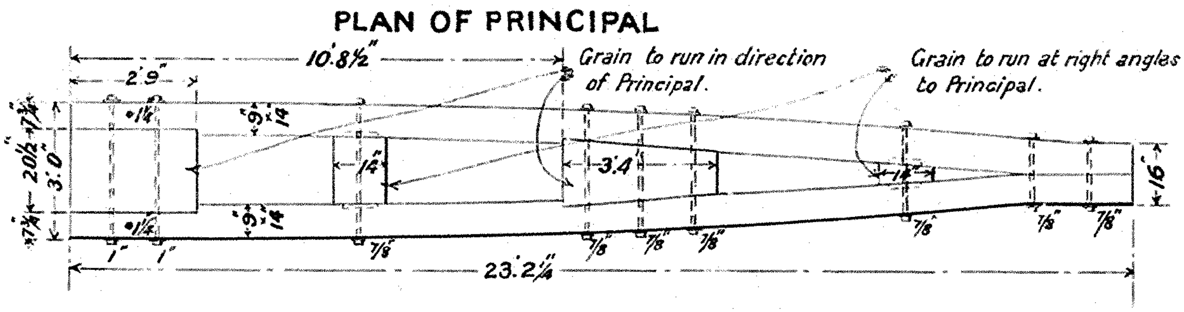


Figure 116: Plan of Principal with Notching and Spacer Orientation in McDonald Truss at Tarraganda

The orientation of the spacers has a significant impact on the behaviour of the assembly. The two larger spacers in a McDonald truss are designed with the grain in the direction of the principal, and these two spacers are notched into the flitches at one end and bolted through. This combination of notching, bolting and parallel grain directions provides a significant level of composite action between the two flitches. However, the two smaller spacers in a McDonald truss principal are designed with the grain at right angles to the principal, with small mortise and tenon connections and a single bolt. The purpose of these spacers is not to provide any composite action, but rather, to maintain a constant distance between the two flitches. Because timber shrinks perpendicular to the grain, only spacers with grain at right angles to the flitches are able to provide a consistent geometry to the member during service life. It is therefore important for the functioning of these members that the orientation of the spacers is preserved.

One common difficulty is obtaining the large spacers, as they must be cut from very large section (up to 520 x 355 mm), heart free, sap free timber. When smaller section timbers are provided the composite action is lost and lateral stiffness greatly reduced. This occurred at Tarraganda, where three timber laminates were used causing serious loss of lateral stability (Figs 117 and 114).

Figure 117: Three Laminates used for Spacer at Tarraganda, Two Laminates with shear keys at Galston

A more successful arrangement was used at Galston (a very short single span McDonald truss), where the single large spacer (460 x 305 mm) was replaced with two smaller spacers connected together with two timber shear keys. This arrangement allowed continued strength and stiffness while also making use of smaller timbers. On three principals at Galston, this was very successfully used. However, on the fourth principal, removal of sapwood caused the shear keys to be protruding, and causing a moisture and debris trap and thereby accelerating deterioration.

Figure 118: Large Spacer replaced with Two Laminates and Shear Keys at Galston – Moisture Trap

Another complexity that arises in the McDonald truss principals is the curvature of the two flitches. The purpose of this bow, as documented by Allan, is to prevent warping and twisting. Bowed members are also used in Allan and de Burgh trusses, but in these later trusses, the largest section to be bowed is 180 mm (7") thick and the bow required is generally 25 mm (1"). In McDonald trusses, 230 mm thick timbers must be bent, and although the total bow is not marked on the drawings, by scaling off the drawings the bow required is in the order of 50 mm.

The design capacity of a single 355 x 230 timber flitch (Fig 116) subjected to permanent bending is 73.6kNm. The bending moment caused by imposing a 50 mm deflection at mid-span of a 7,068 mm length is 69.2kNm. This means that 94% of the capacity it used in fabrication alone.

It is clear that this will not work. It is also clear that the original designers did not design these members for combined bending and compression, but for compression alone. It may be that unseasoned timber was used to fabricate these members, and so the bending stresses would dramatically reduce in the first 12 months of service due to stress relaxation (related to creep). It may be that they were gradually bent into shape, allowing creep to occur over time. It is highly unlikely that seasoned timber was forced into shape, and neither should this occur today.

4.5.2 *Dealing with Permanent Bending in Timber Compression Members*

As noted in the Timber Design Handbook, creep in a member subject to bending occurs due to the inelastic shortening of cells on the compression side of the member. The sum of these microscopic movements can contribute to substantial movement in the member. AS 1720 includes allowances for the increase in deflection with load duration with a duration factor j_2.

The assumptions behind AS 1720.1 which give the values of j_2 given are most appropriate for a uniform moisture environment. Where there is wetting and drying of the timber such as in timber truss bridges, then the creep deformations can be more than twice those given.

Closely related to creep is stress relaxation. Whereas creep involves an increase in strain under constant stress, stress relaxation is the decrease in stress experienced over a period of time by a material subjected to a constant strain. Although AS 1720 includes provisions for creep for serviceability calculations, it does not provide any guidance for the effect that stress relaxation may have on the internal bending stresses in compression members that are constrained to a constant deformation. AS 1720 gives a multiplier by which the deflection can be increased over time while the bending moment remains unchanged. However, it does not give any guidance as to how much the bending moment might decrease over time if the deflection is kept constant.

We know the mid-span deflection of a simply supported beam under a central point load is:

$$\delta = \frac{WL^3}{48EI}$$

If we assume as a conservative estimate that the deflection might increase by 50% in one year, then we would have to reduce the modulus of elasticity by about 33% to maintain equilibrium. We know that the load, the length, the depth and the width remain constant, so the modulus of elasticity is the only parameter in the above equation which can be subject to change with time.

Next, we know that the bending moment due to a centrally located permanent deflection is:

$$M = \frac{48EI\delta}{4L^2}$$

If we apply the same principle, and reduce the effective modulus of elasticity by 33%, then clearly, this results in a reduction in bending moment of 33%. Therefore, even in the case of well seasoned timber subjected to permanent deflections, the bending stress drops in the first year.

If it is the case that the timbers used in bridges were bent to shape in the early stages of seasoning (mc > 25%), and were then subjected to outdoor conditions with large variations in temperature and moisture, a j_2 factor of 1.5 would be overly conservative, and a factor in the order of five would more likely reflect reality. If a j_2 factor of five is used, this corresponds to a reduction in modulus of elasticity of 80%, with a corresponding drop in bending moments of 80%.

The greater the reduction in bending stresses achieved, the greater the capacity for compression stresses remains. It is therefore clear that these curved members must be fabricated from unseasoned timber, and cannot be expected to perform if they are fabricated when seasoned.

4.5.3 Modelling of Diagonals

Like the Old PWD trusses, McDonald truss diagonals consist of a series of single and double members which intersect each other. However, unlike the Old PWD, the McDonald trusses do not include any notching at the point of intersection, but rather, the double members are bowed around the single members and connected with a single bolt. Another difference between the Old PWD and McDonald double diagonals is McDonald's use of structural timber end spacers.

Again, the bending of the two flitches which make up the diagonals will cause significant overstress if seasoned timber is forced into shape. As presented in Table 11, the bending moments due to bowing alone take up 100% of the capacity of the members in the 90' spans, and overstress the members in the 75' spans by 25%. The issue is not as great for the 65' spans.

Table 11: Typical Bowing of Diagonals in McDonald Trusses (values in brackets were also used at times)

	65' Span	**75' Span inner**	**75' Span outer**	**90' Span**
Approximate length	4095 mm	4095 mm	4095 mm	4025 mm
Dimensions of flitches	4" x 9"	4" x 9"	6" x 9"	6" x 9"
Width of intersecting single	4"	6"	6"	6"
With of top packer	3¾"	7¾" (6")	4¾" (5")	5" (4¾")
Total width at top	10¾"	14¼" (12½")	14 ¾" (15")	15" (14¾")
Width of base packer	6"	9½"	6"	7"
Total width at base	12"	16"	16"	17"
Total bow at mid-length	8 mm	36 (48) mm	34 (32) mm	25 (27) mm
Design Bending Capacity	10 kNm	10 kNm	20 kNm	20 kNm
Design Bending Moment	2 kNm	8 (10) kNm	25kNm	20 kNm

The unique aspect of the McDonald truss diagonals is the careful detailing of the double members to provide a high level of composite action between the two flitches. This is achieved by the use of long timber "filling pieces" notched in at each end with the grain of the timber running parallel to the flitches. This arrangement gives substantial resistance against buckling not only to the double members, but also to the single members supported by the double members.

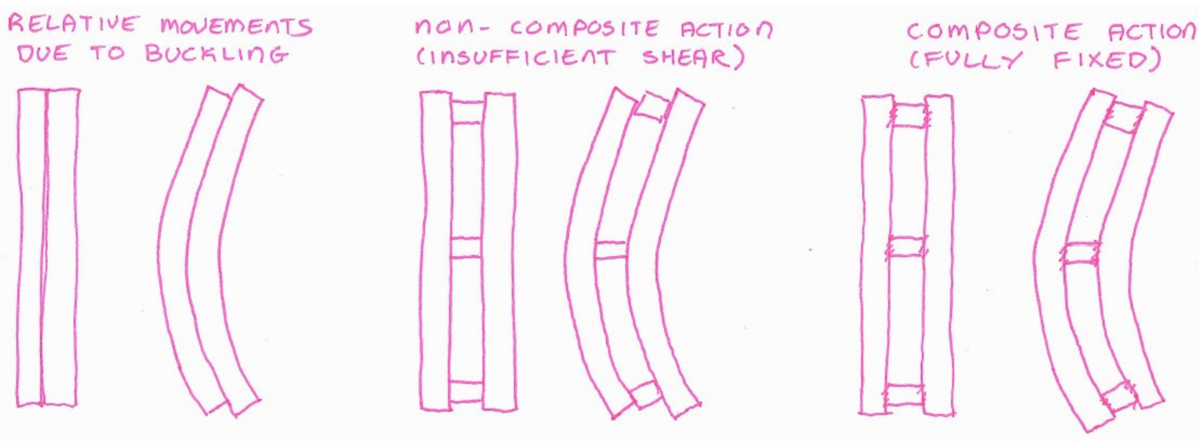


Figure 119: Importance of Shear Stiffness of End Spacers for Buckling Capacity of Double Members

4.5.4 Understanding the Functions of the Cast Iron Shoes

There are two different types of cast iron shoes present in McDonald trusses, and they perform different functions. The first type is found at the connection between the principal and the top chord. This shoe works purely on the basis of clamping action, as the four members (principal, top chord, tension rod and diagonal) all apply compressive stresses to the shoe. The shoe itself is not actually connected to any of the members, although the top of the principal is encased by the shoe on all four sides in order to prevent it from popping out (similar to Old PWD trusses). The tension rods at this point are completely separate from the shoe (unlike Old PWD trusses), and sit on a saddle straddling the top chord. The width of this cast iron shoe is greater than the width of the top chord and principal, whereas the width of other shoes in the McDonald truss are of equal width to the top chord. Another difference between the Old PWD and the McDonald truss is that the four members connected at this point have a common point of intersection. However, whereas in the Old PWD trusses, both the principals and the top chords were cut square to fit into the shoes, in the McDonald trusses, the top chord has to be cut at an angle, and so the top chord is bearing on the cast iron shoe at an angle to the grain, reducing the stiffness. A 6lb (six pounds per square foot) lead sheet is provided on both the inner faces of this shoe.

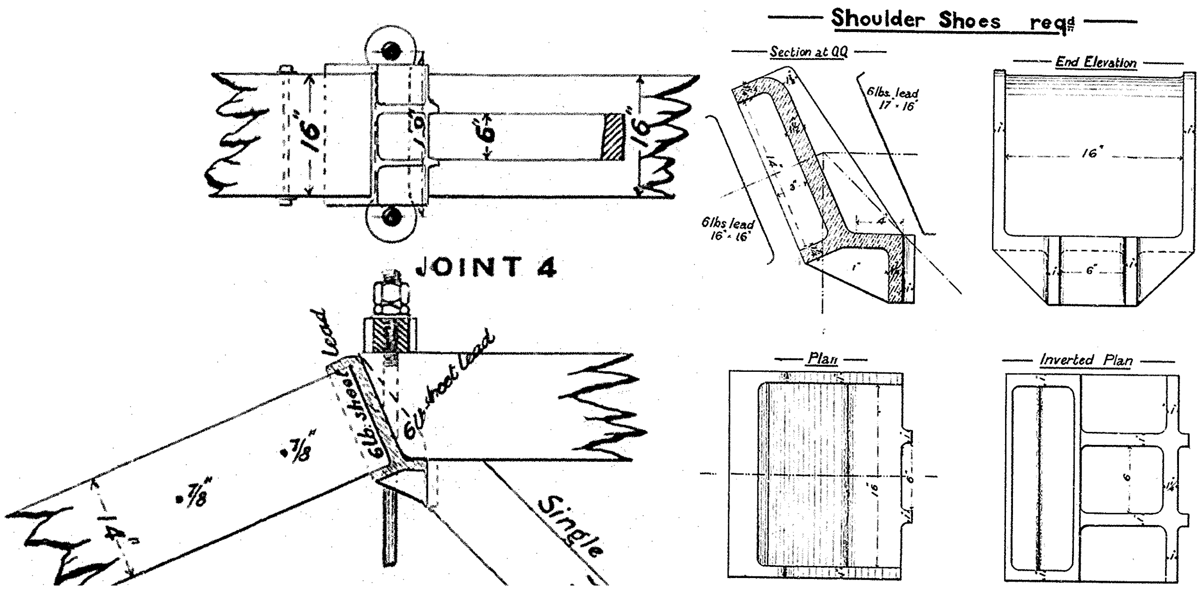


Figure 120: Plan of Cast Iron Shoe at top of Principal in 90' McDonald Truss at Tarraganda

The second type of cast iron shoe connects diagonals to the top chord. These shoes are directly attached to the top chord by two ¾" coach screws screwed into the underside of the top chord. However, the purpose of these coach screws is simply to prevent the shoe from separating from the top chord, as these coach screws have insufficient capacity to resist horizontal loads. The longitudinal loads (from the diagonals in compression) are resisted by shoes being notched into the top chord, and any lateral loads are resisted by very small extensions notched into each side of the top chord. The tension rods pass through the centre of the top chord and the shoes. Diagonals simply bear against the shoe, and are not actually connected to the shoe or the top chord. A 25mm (1") lip is provided in the shoe to prevent the diagonal from slipping out.

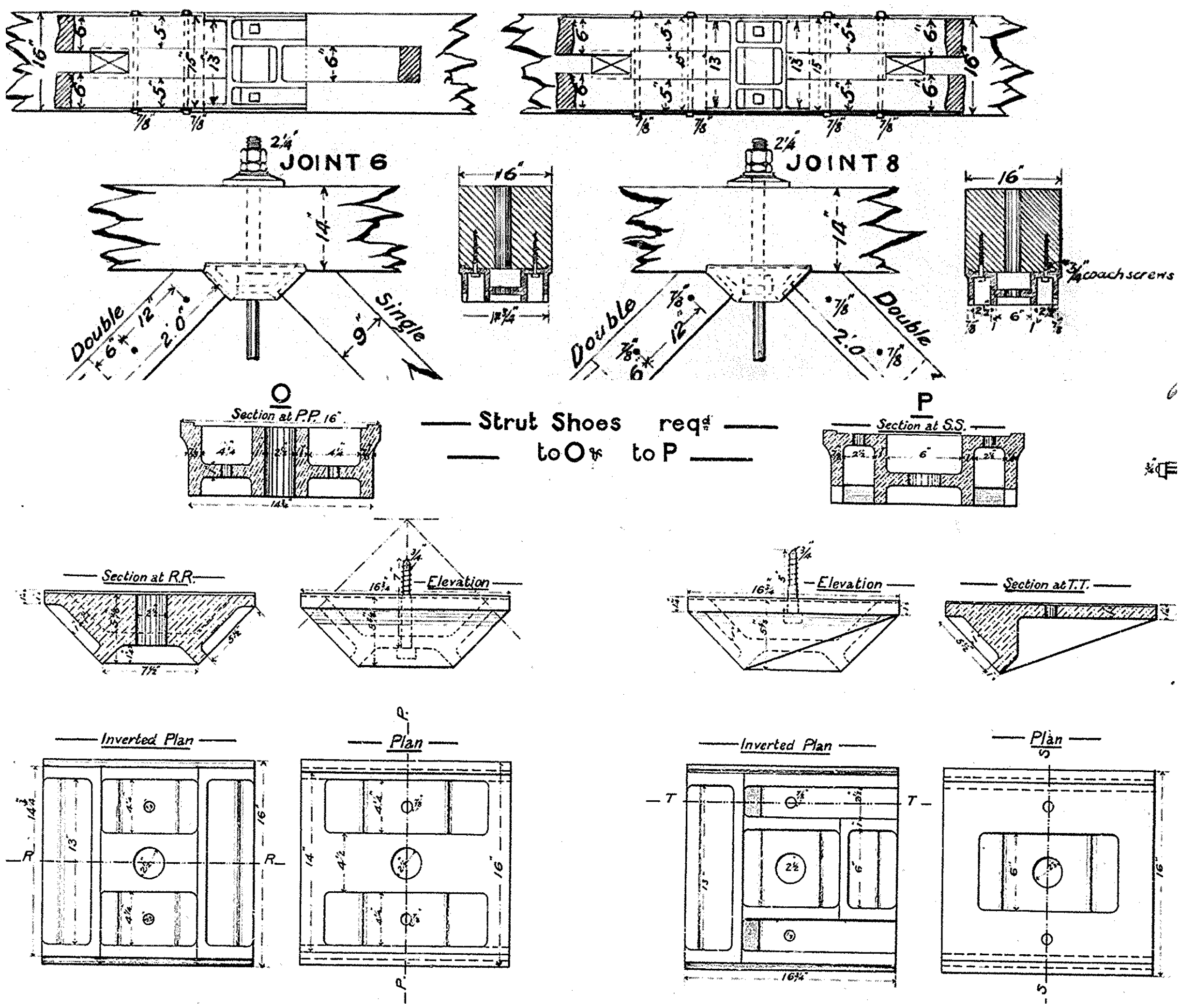


Figure 121: Plan of Cast Iron Shoes at tops of Diagonals in 90' McDonald Truss at Tarraganda

4.5.5 Modelling of Sway Bracing

As noted previously, McDonald designed his top chords to have sufficient capacity to resist buckling without sway bracing, and supplied slender, low capacity bracing only to limit vibrations. However, it has been a common practice, when strengthening these trusses, to attempt to improve the lateral stability by incorporating additional sway braces at each end of the top chord (where originally there were no sway braces as the principal was thought to be sufficiently robust to resist any lateral movements at the ends of the top chord) increasing the section size, and sometimes also increasing the angle of bracing by increasing the length of the cross girders.

The original sway bracing in all McDonald trusses (65', 75' and 90' spans) consisted of 6" x 4" x ½" T-irons at each internal panel point, connected with two ¾" bolts at each end. The connections as originally designed are able to resist either a tension or compression force in the sway bracing of no more than approximately 20kN. The top bolts are acting in shear perpendicular to the grain of the timber, which limits their capacity, and the bottom bolts are too close to the ends of the timber cross girders, and quickly become ineffective due to deterioration.

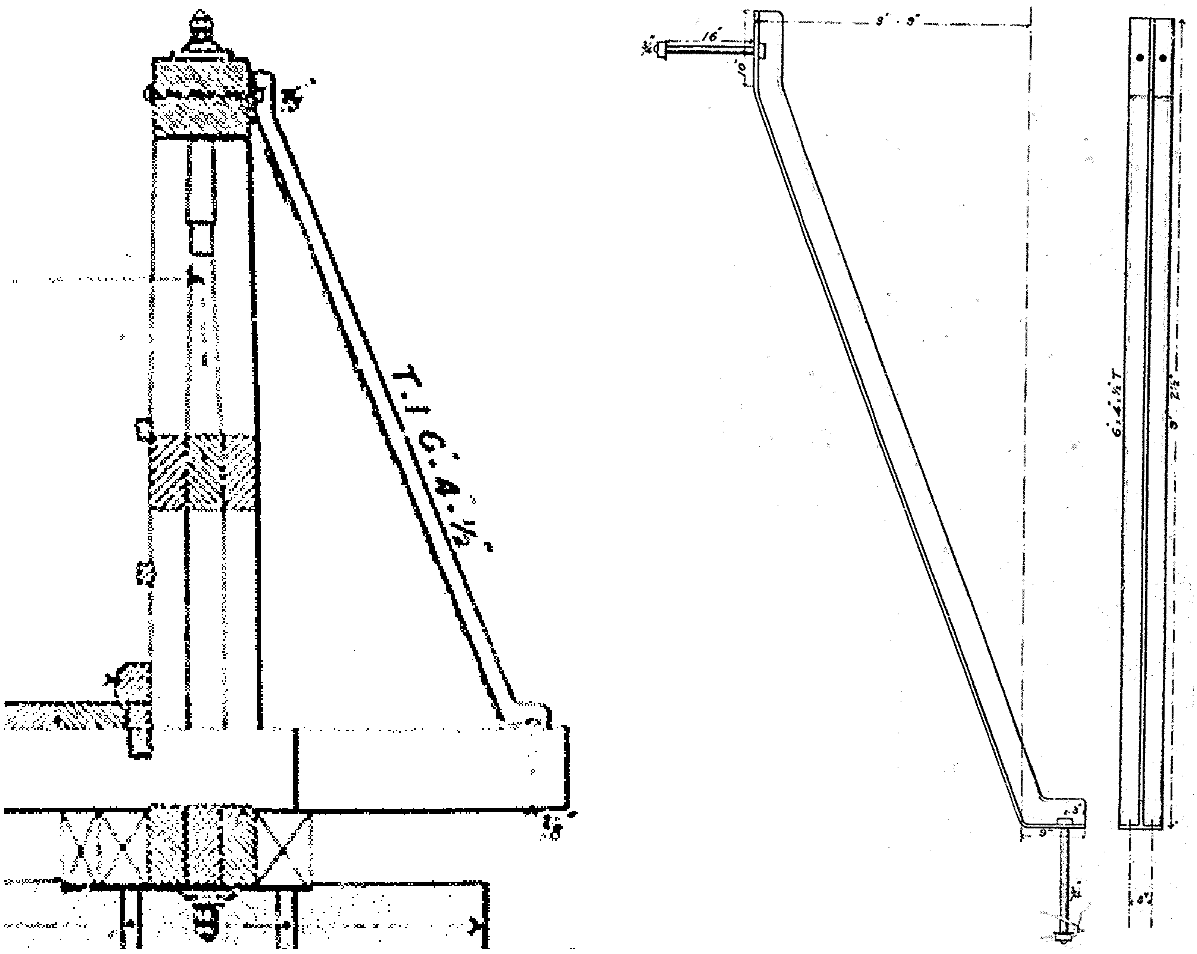


Figure 122: Plan of Sway Bracing for 90' McDonald Truss at Tarraganda

A tension or compression force of 20kN in the sway bracing is able to provide a lateral restraint to the top chord of only 8kN. According to the calculations provided in Table 12, the lateral restraint required for the design forces considered is approximately eight times this figure. Increasing the number of bolts penetrating the top chord will not significantly increase the capacity of the sway bracing to provide sufficient restraint. The problem is that bolts in timber acting in shear perpendicular to the grain have inherently low strength and cannot be relied upon for forces beyond approximately 20kN. Where larger capacities are required, a different connection must be used. Unless a different connection is used, the sway bracing cannot be relied upon in modelling or in calculations for providing lateral restraint to the top chord.

Table 12: Approximate Calculations for Lateral Restraint required for McDonald Truss Top Chords

$$N^*_R = \frac{0.1N^*_c}{n+1} h_{26} h_{27} g_{38} \text{ [AS 1720.1-2010, Clause E7.2.3]}$$

	65' Span	75' Span	90' Span
N^*_c *(design force* LL+DL*)*	950 kN	1250 kN	1550 kN
n (number of restraints)	1	2	3
$h_{26}h_{27}g_{38}$ *(empirical factors)*	1.5	1.5	1.5
N^*_R *(lateral restraint required)*	70 kN	65 kN	60 kN

4.5.6 *Effects of Using Shorter or Smaller Timber*

McDonald was still designing his truss type in a time when timber resources were relatively plentiful and cheap, and so, like Bennett, he used very large and long sections of timber. Unfortunately, timber resources are no longer plentiful or cheap, and so modifications have been made to these trusses attempting to make use of shorter lengths and smaller section sizes.

The use of shorter lengths in the laminated timber bottom chord has a detrimental effect on the strength and stiffness of this member as is the case for the Old PWD trusses. Again, due to the difficulties in obtaining long large sections of timber, some top chords have been replaced with two smaller members bolted together. Despite the large number of bolts generally used to connect the two pieces together, the bolted connection is significantly less stiff than a single member. Not only does this reduce the compression capacity of the top chord element, but it significantly reduces the lateral stability of the truss due to uneven bearing at the top chord ends. It has already been noted that the use of smaller sections for the spacers in the McDonald truss principals has a negative impact on the strength, stiffness, and stability of these members.

4.5.7 *Summary*

McDonald trusses, like Old PWD trusses are susceptible to second order effects. For this reason, it is important to analyse the structure using a non-linear second order analysis which takes into account not just the compression-only and tension-only members, but also the three dimensional effects of the truss under various load combinations. Modelling must take into account not only the member and material properties, but also the connections.

Where original lengths and sections of timber are no longer available, the negative impacts of using smaller section timbers must be taken into account in the modelling. This is likely to occur for laminated timber bottom chords, where the maximum length was 16.3m (heart free, sap free with cross section of 355 mm x 115mm), and it is also likely to occur with top chords, where lengths in excess of 13 m are required with sections of heart free timber 355 mm x 405 mm.

While diagonal members in Old PWD trusses depend purely on bearing action at the base for their connections, they are at least bolted to the top chord. However, in McDonald trusses, the diagonal members are not connected to either the top or the bottom chords, but rely on bearing against cast iron shoes at the top and against wedges bearing against the cross girders at the bottom. This truss is therefore very susceptible to gaps opening up between members.

Figure 123: Gaps opening up in McDonald Truss Connections (Galston and Junction Bridges)

4.6 Modelling Allan Trusses

Unlike the previous truss types, Allan trusses depend on pure truss action (no redundancy, no alternate load paths), rather than a combination of truss and arch behaviour. This means that their structural behaviour is more easily understood and more easily modelled. Allan also designed specifically to maximise durability and to allow member replacement for minimal cost. He introduced camber in his timber decks and large gratings at ends of spans to permit the quick escape of water and to prevent water lying in pools on the surface of the timber decking planks.

The most significant structural change in the Allan truss is the fact that all of the timber compression members are at the same angle. This means that there is a greater variation in stress in the top and bottom chords along the lengths of the chords (less towards the piers, and greater towards mid-span). It also means that the horizontal thrust imposed on the bottom chord by the principals is greatly reduced, which allowed Allan to use cast iron shoes to transfer the loads rather than the large timber butting blocks with heavy bolts that were previously necessary. This also allows structural separation between spans, so there is no continuity of the bottom chord over the piers, and there is no provision for the timber approach spans to provide secondary load paths for the thrust from the principals. Another great advantage of having all the braces at the same angles is that any shrinkage in the timber can be taken up by means of the suspension rods.

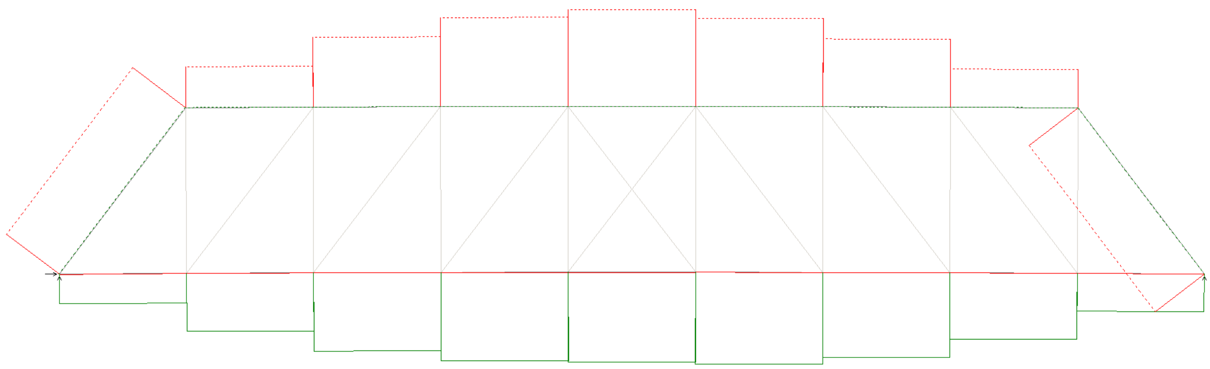


Figure 124: Axial Force Envelope Showing Variation in Top and Bottom Chords in Allan Truss

Dare wrote, "The standard timber-truss employed for many years was one having timber chords and braces, and vertical suspension-rods. In the bridges of this type designed by Mr. Percy Allan, M. Inst. C.E., between 1893 and 1899, many improvements were introduced, and these bridges will doubtless prove more durable and less costly to maintain than their predecessors. It has been found, however, that in almost every case the timber lower-chord has been the first member of the truss to fail, and the flitches, being in tension, are very difficult to replace."

Bottom chords in Allan trusses are not theoretically the first mode of failure, but they are very sensitive to localised deterioration, especially at connections. For this reason, the timber bottom chord is often replaced with steel bottom chords with a non-structural timber facia added. This is relatively uncomplicated in Allan trusses due to the consistent geometry, which allows for thermal movements in the bottom chord more difficult to accommodate in earlier truss forms.

4.6.1 Behaviour of the Top Chords

Due to the fact that Percy Allan wanted to be able to provide footways on some of his bridges, and the sway bracing in the older truss types was a source of inconvenience in this matter, Allan decided to design the top chords in his 70' and 90' trusses as columns with a varying load, unsupported in the lateral direction. In fact, the earliest Allan truss drawings have no sway bracing shown at all, but have "wind stays" instead, detailed as shown in the figure below. However, sway bracing was soon introduced, even if it did not appear on the drawings. For example, the drawings for the Allan trusses at Swan Hill show no sway braces, but the photographs at the opening of the bridge clearly indicate that sway bracing has been provided. Sway bracing was incorporated into the later standard drawings for 70' and 90' spans.

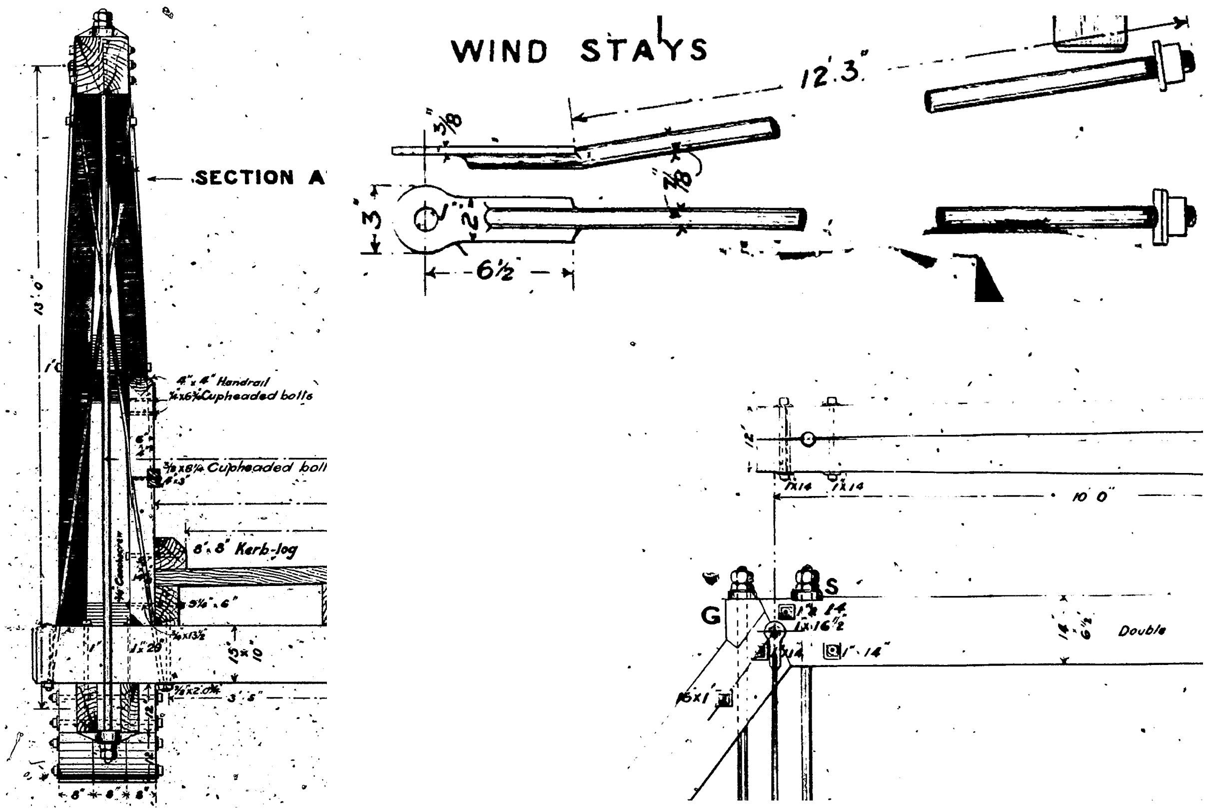


Figure 125: Original Detailing of Wind Stays and no Sway Bracing on Allan Trusses

The standard sway bracing in Allan trusses (70' and 90' spans) consisted of 4" x 2" x 3/8" T-irons (or 4" x 4" x 3/8" T-irons) at some or all panel points, connected to the top chord with two ¾" bolts at end panels and with two ¾" coach screws for the intermediate sway bracing. Sway bracing is connected to the timber cross girders with two ¾" bolts. Unlike the McDonald trusses, bolts connecting Allan truss sway bracing are staggered both at the top and at the base.

The connections as originally designed are able to resist either a tension or compression force in the sway bracing of no more than approximately 15kN. The top connectors are acting in shear perpendicular to the grain of the timber, which limits their capacity, and the staggering of bolts causes them to provide restraint against shrinkage for the timber, again limiting their capacity.

A tension or compression force of 15kN in the sway bracing is able to provide a lateral restraint to the top chord of only 6kN. According to the calculations provided in Table 13, the lateral restraint required for the design forces considered is more than five times this figure.

Table 13: Approximate Calculations for Lateral Restraint required for Allan Truss Top Chords

$$N^*_R = \frac{0.1N^*_c}{n+1} h_{26} h_{27} g_{38} \quad \text{[AS 1720.1-2010, Clause E7.2.3]}$$

	70' Span	90' Span
N^*_c *(design force* LL+DL*)*	1400 kN	1700 kN
n (number of restraints)	4	4 (or 8)
$h_{26}h_{27}g_{38}$ *(empirical factors)*	1.5	1.5
N^*_R *(lateral restraint required)*	42 kN	50 kN (or 30 kN)

As is the case for the McDonald trusses, the sway bracing in Allan trusses cannot be relied upon in modelling or in calculations for providing lateral restraint against buckling to the top chord. The figures below show that top chords buckle without regard for the sway bracing, even when sway bracing is provided at all panel points. Strengthening the sway bracing does not achieve increased lateral restraint unless connections both at the top and at the base are also considerably modified. There are therefore only two options available for modelling the top chord. Either the top chord is analysed as a laterally unrestrained column with a varying load, or the sway bracing connections are modified so that adequate lateral restraint is provided. Allan was particularly careful in his designs to intersect both primary and secondary members at a common point so that no secondary bending stresses were caused due to a lack of accurate triangulation.

Figure 126: Lateral Buckling of Top Chord in Allan Trusses (Thornes Bridge and Euminbah)

4.6.2 Lateral Restraint: How Timber Trusses Stay Up

In the past it has generally been assumed that all connections between the diagonal or vertical timber compression members and the top and bottom chords are best modelled as pinned. Indeed, this was the recommendation from the testing carried out in the early 1990s as part of the Euminbah Bridge experiments where an Allan truss top chord connection was tested. If a two dimensional model is used to obtain design forces then this issue may not initially concern the designer. However, if a three dimensional model is used and if all connections are pinned, then the truss becomes unstable and falls sideways unless the sway bracing is also modelled.

However, as is discussed in the previous section, the original design of the sway bracing in all but the Old PWD trusses is such that it is incapable of carrying any significant load, and therefore cannot provide lateral restraint to top chords. If this is the case, how do the trusses stay up?

Well, Old PWD trusses were originally designed with rather large timber sway braces at every panel point with connections very carefully detailed to provide adequate load transfer in both tension and compression. McDonald, however, went away from this approach, and instead relied entirely on the significant stiffness of the principals to provide lateral stability to the truss.

Allan, de Burgh and Dare all used castings for connections which were sufficiently stiff for the truss to stay up unassisted. It is the fixity of these connections which give stability to the trusses.

Figure 127: Construction of Barham Bridge with Vertical Struts Standing Unassisted.

One example showing the extent of the fixity of a bottom chord to diagonal connection can be seen at the old Dare truss near Moonan Flat. Here the bridge has been severely damaged by a flood which has caused bending of the bottom chord, yielding of one sway brace, bending of a tension rod, and breakage of a timber compression diagonal. A new concrete bridge has now been constructed next to the original bridge. The interesting thing to note is that the bottom half of the timber diagonal still stands in close to its original position even after such an impact.

Figure 128: Impact Damaged Dare Truss near Moonan Flat

4.6.3 Modelling Compression Members with Timber Spacers

Although the packing blocks in spaced timber compression members may appear similar to batten plates in steel columns, the design of timber spaced columns is very different. Unless they are glued in place, the packing blocks in timber columns are not sufficiently rigid to enable the column to act as a unit. Even if they are tightly bolted when the structure is first erected, the inevitable shrinkage will cause subsequent looseness, with a great reduction in strength.

Another complexity that arises is the curvature of the two flitches. The purpose of this bow, as documented by Allan, is to prevent warping and twisting. Bowed members are also used in McDonald and de Burgh trusses. Refer to Section 3.5.2 for dealing with fabricated curvature.

Buckling strength of a timber member is a function of a great number of complex parameters. These include those used for material failure and creep criteria, material variability, nonlinear material characteristics, the random dispersion of strength reducing characteristics and initial crookedness. Knots in timber and material variability make it impossible to load a member with perfectly concentric axial loads. Although there are provisions for spaced columns in AS1720.1-2010, these provisions do not cover the kinds of spaced columns used in timber truss bridges.

The provisions of AS1720.1-2010 Clause E4 shall not be used for heritage timber truss bridges.

4.6.3.1 Distribution of Load between Two Flitches

In 2007, RMS did some load testing and strain gauging of the 90' Allan truss bridge at Vacy in the Hunter Region. Among other things, the load sharing between the two flitches of the spaced columns comprising the principals, diagonals and top chords of the truss was investigated. Results varied substantially with some members displaying almost equal strains in the two flitches (45% / 55%), while other members had differences in strain of up to 30% / 70%. Significantly, it was not always the inner flitch or the outer flitch that showed the highest strains, which suggests that load sharing is dependent upon the variation in properties between the two flitches that make up a particular member as well as the global effects in the truss.

Reasons for unequal load sharing between the two flitches may include lack of fit (if one is slightly longer than the other the longer will take more load), variability in modulus of elasticity (the stiffer member will attract the greater load), or out of straightness (if one member has a different curvature to the other, more load will be taken by the straighter member).

For double tension members, each element must be designed to take 60% of the load. However, for compression members the unequal distribution of load between two flitches has a negligible effect on the buckling capacity of the assembly as a whole. This is because the unloaded flitch is still able to provide substantial restraint against buckling to the loaded flitch through the timber spacers. Even if one flitch is taking 100% of the load due to one of the abovementioned causes, the capacity of the assembly is still the same as it would have been if each flitch was sharing exactly 50% of the total load. It is therefore not necessary to make any reductions in capacity to take into account the effects of unequal distribution of load between two flitches in compression.

4.6.3.2 Spaced Column Theory and How Heritage Timber Trusses Compare

Providing composite action between two compression members can have significant structural advantages. In order for composite action to be effective, the relative displacements that occur between the two flitches (maximum at the ends, and zero at the centre) must be restrained. For example, the theoretical buckling capacity of a single rectangular column is determined by:

$$N = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 Edb^3}{12L^2}$$

If two identical columns are provided (shown in the first illustration of Fig 129), then the capacity is doubled. However, if the two columns are made composite (with no spacers) then the effective b has doubled, and so the capacity is increased eight fold (2^3). This means that double composite columns have four times the capacity of double non-composite columns.

This effect is even greater when spacers are introduced. For the non-composite case (shown in the second illustration of Fig 129), the capacity is double that for a single column. However, for the fully composite case, the second moment of area (I) is considerably increased. Say, for example, that timber spacers are provided at the same width as the timber flitches. This would give an effective second moment of area (I) of 26 times that of a single column. In turn, this means that the composite spaced column has thirteen times the capacity of the non-composite.

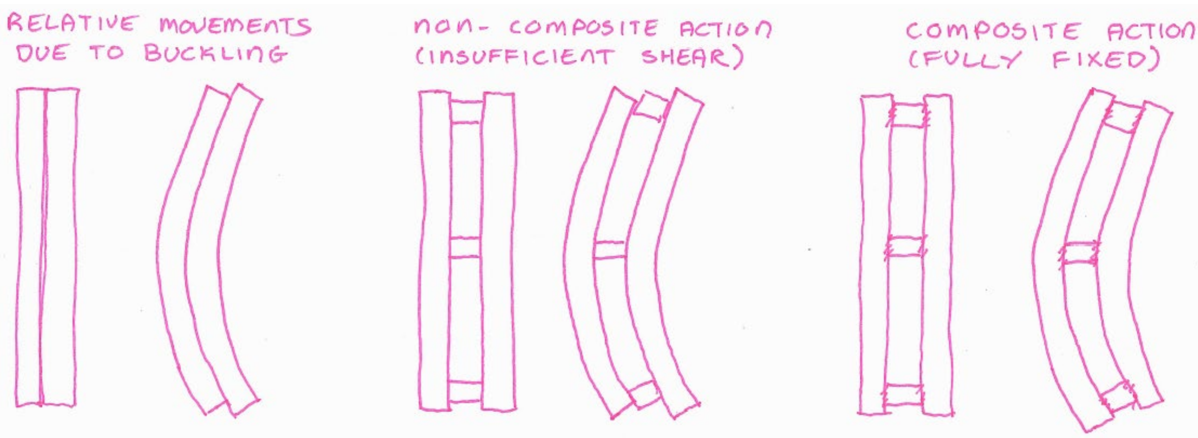


Figure 129: Importance of Shear Stiffness of End Spacers for Buckling Capacity of Double Members

It is important to recognise that it is impossible to achieve fully composite action in timber spaced columns. Connections in timber are never entirely rigid, the timber flitches have a low modulus of elasticity perpendicular to the grain, and timber spacers also have some flexibility. The assumption made in the Australian Standard is that differential movement between the two flitches is restrained by the nails or bolts transferring the shear loads from one flitch to the timber spacer, and then the timber spacer transfers the shear loads to the nails or bolts on the other side of the spacer, which then transfer the shear load to the other timber flitch. This behaviour is not reflected in timber truss bridges due to the size of the connectors (large bolts) and the weakness of the timber spacers. Shear forces tend to simply split the timber spacer, and so the only shear stiffness that remains is the shear stiffness of the bolts spanning between the two flitches.

4.6.3.3 Behaviour of Bolted Timber Spacers Subjected to Shear Loading

Allan clearly marked the direction of grain of the timber spacers to be at right angles to the timber flitches making up the diagonal compression members (including principals) in his trusses. For the 90' spans he often detailed a mortise and tenon joint for the central spacer only in the principals, but this detail was not used for the 70' spans. As previously discussed, the critical structural issue is the behaviour of the bolt and spacer assembly at the end of the member.

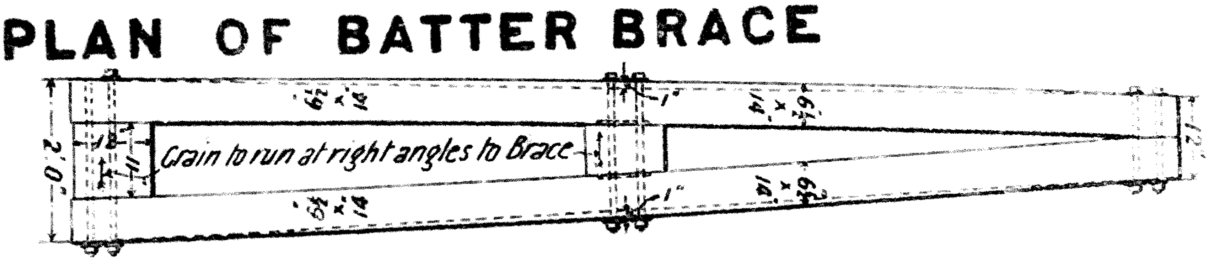


Figure 130: Plan of Principal with Spacer Orientation in 90' Allan Truss at Vacy

Laboratory testing was conducted in 2011 to investigate the behaviour of these assemblies under shear loads. Full size, seasoned bridge timbers were tested. It was discovered that the timber spacers provide almost zero shear resistance, as they split under very low loads. This is the primary reason that the spaced column provisions of AS1720.1 should not be applied to these trusses. The code has its primary focus on relatively small section timbers (marketable sizes) connected with relatively small connectors (nails, screws and bolts up to 16 mm in diameter). Although the behaviour displayed in the first diagram in Fig 131 may be expected if connectors are nails or very small diameter (say M6) bolts, this does not occur with larger bolts. This is because the relative stiffness of the bolts and the timber is significantly different in the case of large diameter bolts. Hardwood timber loaded perpendicular to the grain by nails or very small diameter bolts will cause the connector to bend, but with large bolts, the timber simply splits.

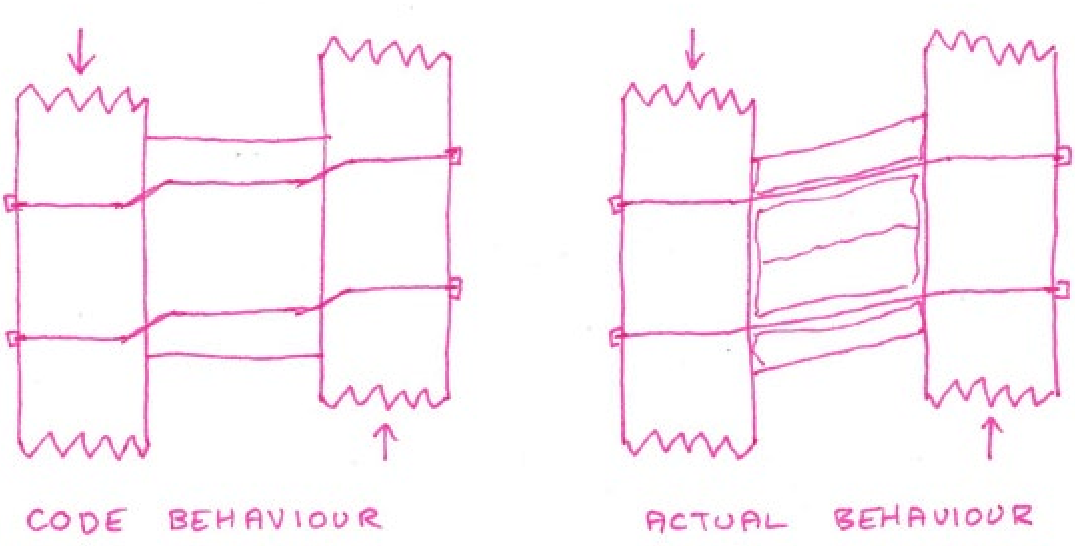


Figure 131: Comparison of Code Behaviour with Actual Behaviour of Timber Truss Spacers

4.6.3.4 Elastic Critical Buckling Load

In order to accurately model the stiffness of a member in an Allan truss, both the modulus of elasticity (E) and the second moment of area (I) must be given. For truss timber, the relevant modulus of elasticity is 16,000MPa (F22 timber). However, the second moment of area for an assembly of two members is more complex. If zero composite action is assumed, the model will underestimate the stiffness which may cause member buckling and instability in the model. However, if full composite action is assumed then the model will overestimate the stiffness.

In order to determine a reasonable value for the second moment of area, the column assembly can be modelled in Microstran, and an elastic critical buckling analysis undertaken. This will give an elastic critical buckling load N, which can then be used to determine an equivalent value of I.

As discussed previously, the timber spacers do not provide any shear resistance, and so they should not be included in the Microstran model. However, the model should accurately reflect the distance between the two flitches, and the full number of bolts which connect the two flitches together. Supports should be pinned at each end, and load applied directly to the flitches.

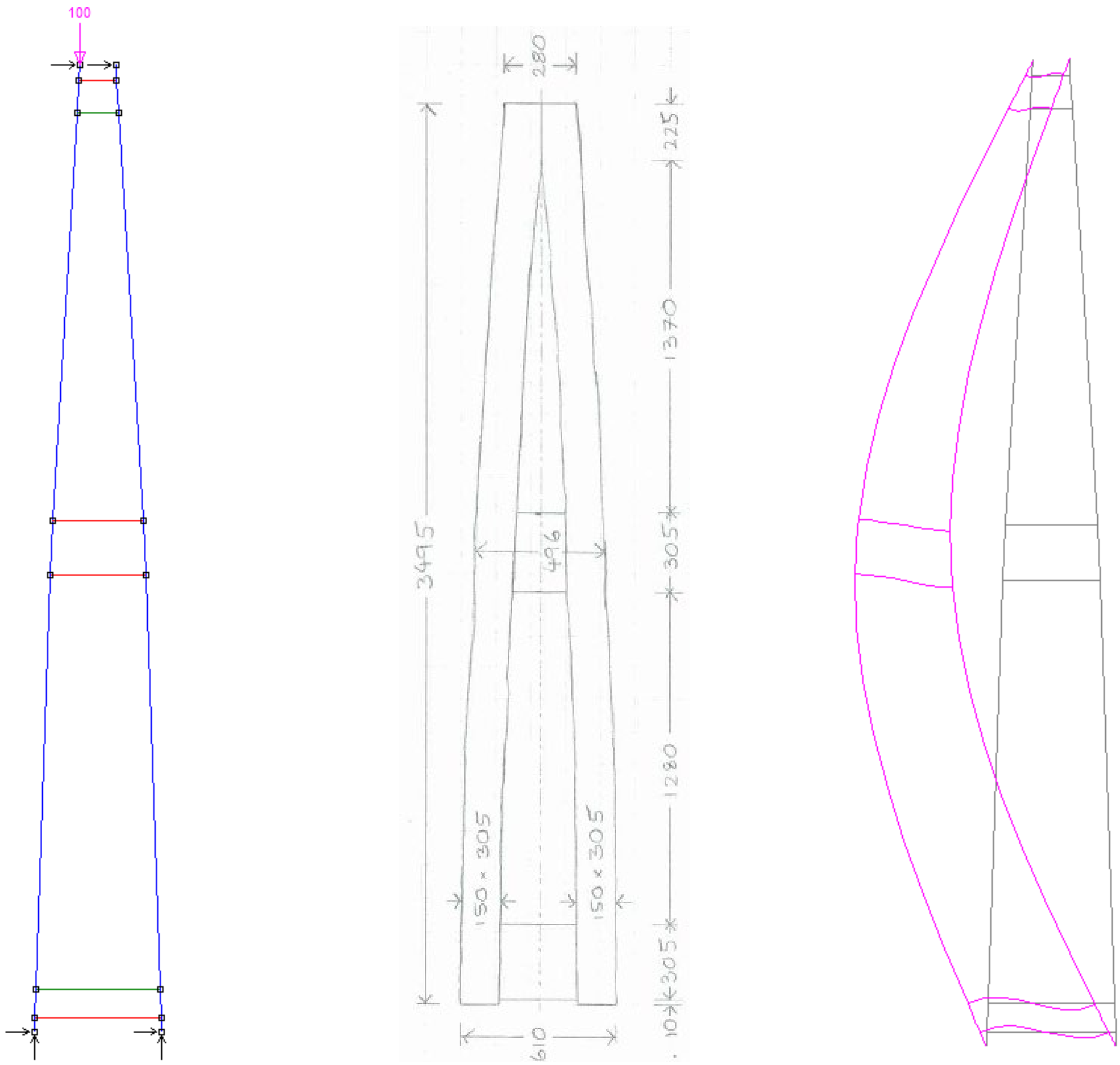


Figure 132: Model and Elastic Critical Buckling Analysis of 70' Allan Truss Principal

Equivalent values of I can be now calculated as given in Tables 14-16 below. These tables are not to be used for determining the capacity of members, but for determining equivalent values of the second moment of area to input into a global model of the bridge. For determining the capacity of a compression member the lower 5th percentile modulus of elasticity must be used.

Table 14: Equivalent Second Moment of Area for Modelling 70' Allan Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Principal	Brace	Brace	C-Brace	C-Brace
depth of flitches	150 mm	115 mm	115 mm	100 mm	75 mm
width of flitches	305 mm	200 mm	200 mm	200 mm	150 mm
length of assembly	3495 mm	3620 mm	3620 mm	3620 mm	3660 mm
I (Non-Composite)	170×10^6mm^4	$50\ \times10^6$mm^4	50×10^6mm^4	35×10^6mm^4	10×10^6mm^4
N (Non-Composite)	2215 kN	610 kN	610 kN	400 kN	125 kN
N (Microstran Result)	2705 kN	985 kN	1040 kN	940 kN	390 kN
I_x (Equivalent)	210×10^{-6}m^4	80×10^{-6}m^4	85×10^{-6}m^4	80×10^{-6}m^4	35×10^{-6}m^4
I_y (=2bd^3/12)	710×10^{-6}m^4	155×10^{-6}m^4	155×10^{-6}m^4	135×10^{-6}m^4	40×10^{-6}m^4

Table 15: Equivalent Second Moment of Area for Modelling 90' Allan Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Principal	Principal	Brace	Brace	C-Brace
depth of flitches	180 mm	165 mm	125 mm	115 mm	100 mm
width of flitches	355 mm	355 mm	200 mm	200 mm	200 mm
length of assembly	4250 mm	4250 mm	4345 mm	4345 mm	4345 mm
I (Non-Composite)	345×10^6mm^4	$265\ \times10^6$mm^4	65×10^6mm^4	50×10^6mm^4	35×10^6mm^4
N (Non-Composite)	3015 kN	2320 kN	545 kN	425 kN	280 kN
N (Microstran Result)	3410 kN	2705 kN	920 kN	775 kN	600 kN
I_x (Equivalent)	390×10^{-6}m^4	310×10^{-6}m^4	110×10^{-6}m^4	90×10^{-6}m^4	70×10^{-6}m^4
I_y (=2bd^3/12)	1340×10^{-6}m^4	1230×10^{-6}m^4	165×10^{-6}m^4	155×10^{-6}m^4	135×10^{-6}m^4

Table 16: Equivalent Second Moment of Area for Modelling 110' Allan Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Principal	Brace	Brace	Brace	C-Brace
depth of flitches	150 mm	150 mm	100 mm	100 mm	100 mm
width of flitches	355 mm	305 mm	255 mm	255 mm	255 mm
length of assembly	7220 mm	7300 mm	7340 mm	7340 mm	7340 mm
I (Non-Composite)	200×10^6mm^4	$170\ \times10^6$mm^4	40×10^6mm^4	40×10^6mm^4	40×10^6mm^4
N (Non-Composite)	605 kN	510 kN	125 kN	125 kN	125 kN
N (Microstran Result)	1585 kN	1435 kN	445 kN	510 kN	535 kN
I_x (Equivalent)	525×10^{-6}m^4	485×10^{-6}m^4	150×10^{-6}m^4	175×10^{-6}m^4	180×10^{-6}m^4
I_y (=2bd^3/12)	1120×10^{-6}m^4	710×10^{-6}m^4	275×10^{-6}m^4	275×10^{-6}m^4	275×10^{-6}m^4

4.6.4 Understanding Connections

In Allan trusses, the horizontal forces from all diagonal members, including principals, are taken up by means of castings, having lugs (or shear keys) notched into the chords, and where two lugs are necessary it will be noticed that the deeper lug is at the back of the casting, so as to distribute the horizontal load over a larger area and reduce the risk of failure by shearing between the lugs.

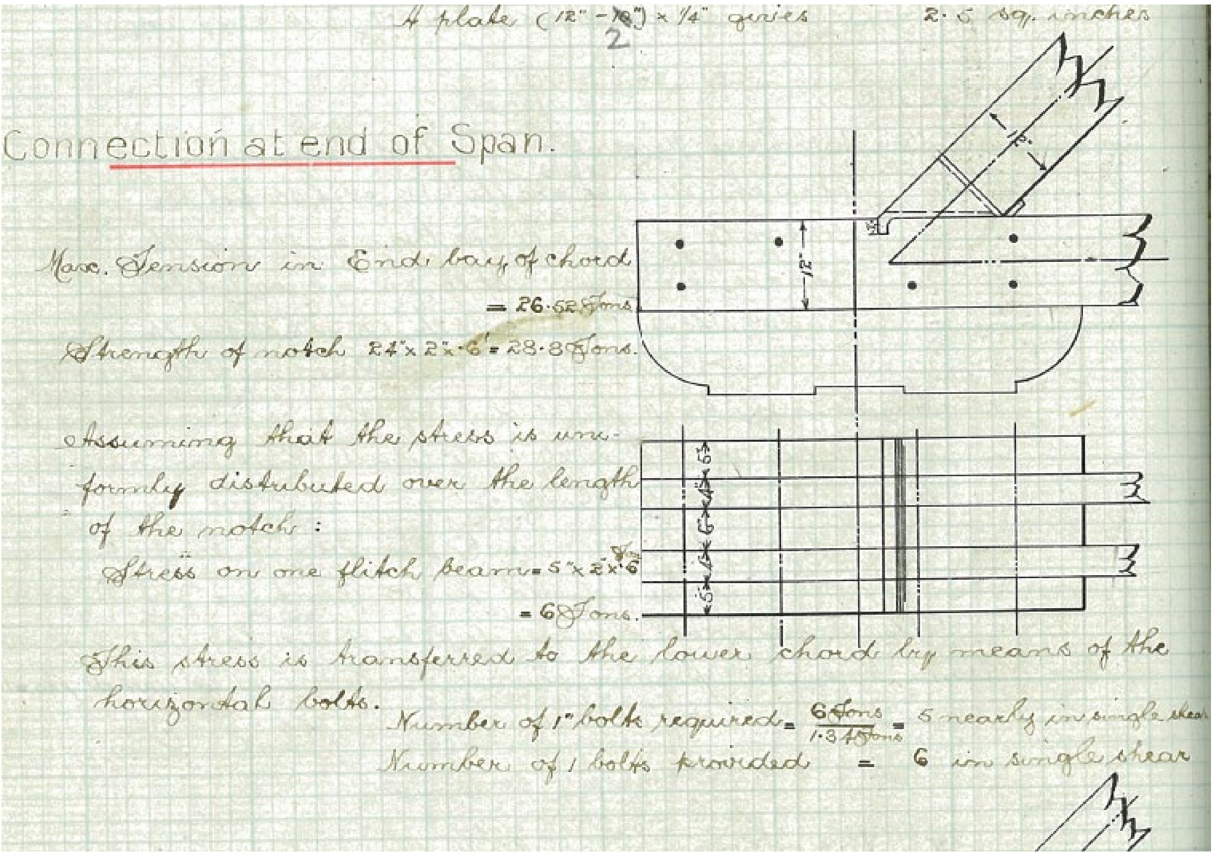


Figure 133: Excerpt from Percy Allan's Book of Calculations – p225 Calculations for 70' Allan Truss

Principals have larger stress, and a deeper shear key (2") acts over more timber members (five rather than two). As noted in Allan's calculations, the design relied upon the stress being transferred from the additional timbers to the lower chord by means of the horizontal bolts.

This does seem to go against the theory he used when designing the bottom chord splice plates, where he commented that, "Following American practice, the bolts passing through cover plates are not in any way relied upon, being simply provided to keep the plates up to their work…" The stiffness of a shear key is considerably greater than the stiffness of a bolted connection, and so the shear key from the cast iron shoe at the base of the principal is likely to impose a greater shear force on the bottom chord flitches than on the other three timber members. It may be for this reason that de Burgh designed a strengthening of the principal shoes at the 110' Allan Truss at Morpeth, in which he provided an additional shear key, the details of which are shown in Fig 134. Allan noted that he did not rely on friction between the timber and the metal, although he thought it was likely to be present and to reduce the actual stresses experienced by the shear keys.

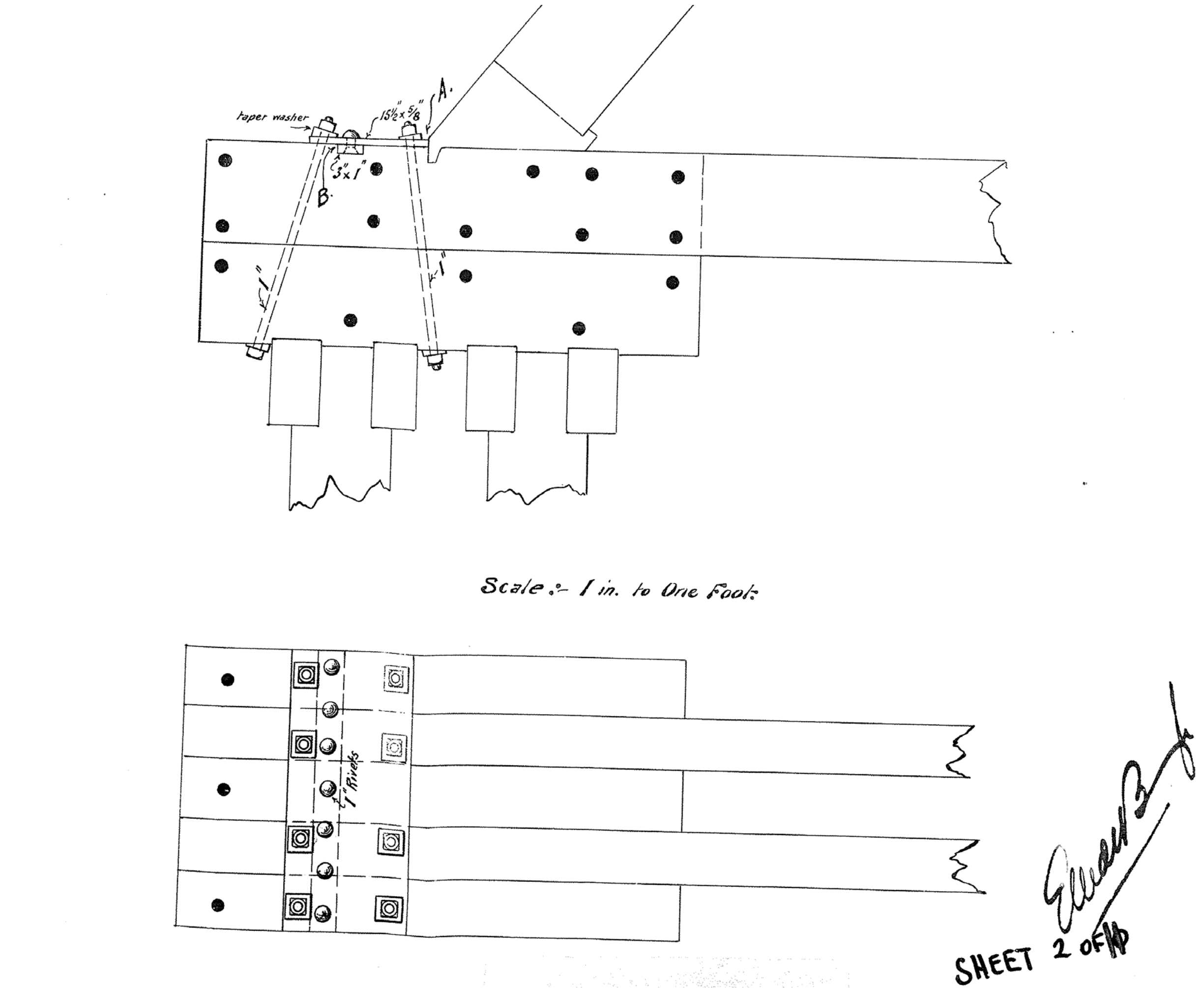


Figure 134: de Burgh's strengthening of the Principal Base Shoe on the 110' Allan truss at Morpeth

Like the McDonald trusses, the top shoes in an Allan truss are coach screwed into the underside of the top chord, and the purpose of these coach screws is simply to prevent the shoe from separating from the top chord. The longitudinal forces (from the diagonals in compression) are resisted by shear lugs being notched into the top chord. Unlike the McDonald trusses, diagonal members do not simply bear against the shoe, but are also bolted with a single bolt to the top and bottom shoes. This prevents the timber from slipping away from the shoe under loads.

The effective length of a timber compression member is found by modifying the actual length by an effective length factor (g_{13}) which is obtained from Table 26. It is generally assumed in truss analysis that both ends are restrained in position only, giving a g_{13} factor of 1.0. However, as discussed in Section 4.6.2, there is significant fixity in the connections of the later truss types, particularly around their minor axes. A g_{13} factor of 0.7, correlating to a member fixed at both ends, is therefore appropriate. This factor, used in both the timber and the steel design standards, has significant conservatism due to the fact that real connections are never fully fixed or fully pinned. This factor gives a result which is equivalent to a member being fixed at the base and pinned at the top (which is what should be modelled in an elastic critical buckling analysis).

Cast iron has very good resistance to corrosion, but is a brittle material, and cast iron shoes have a tendency to fracture in Allan trusses as much as in the earlier Old PWD and McDonald trusses:

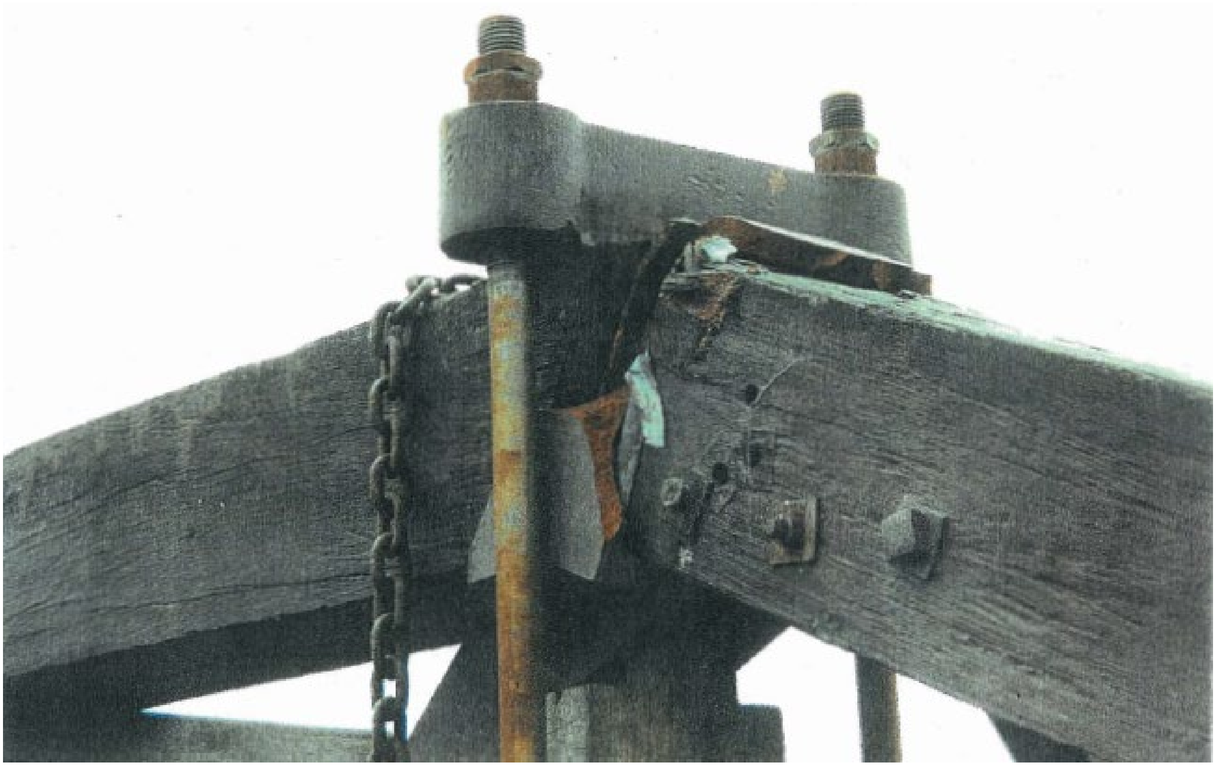


Figure 135: Brittle Fracture of Cast Iron Shoes in McDonald Truss at Tumut

Figure 136: Brittle Fracture of Cast Iron Shoes in Allan Truss at Morpeth

There is some variability in shoe fabrication between Allan trusses. For example, Beryl Bridge shoes have very distinctive eye holes for the coach screws, whereas Wallaby Rocks has a more typical squared shape shoe. It seems this was a difference in fabrication rather than design.

Figure 137: Detail of Top Chord to Principal Connection at Beryl Bridge

Figure 138: Detail of Top Chord to Principal Connection at Wallaby Rocks

4.7 Modelling de Burgh Trusses

Like the Allan trusses, de Burgh trusses depend on relatively pure truss action and their structural behaviour is relatively easily to understand and model. Apart from the obvious change of truss shape (with vertical compression members and diagonal tension members), the most significant structural change in the de Burgh truss is the fact that all of the flitches in the compression members are orientated with their major axes at 90° to the top and bottom chords. The connection of the diagonal steel tension rods with the bottom chord is pinned, and the tension rods have eyes welded onto one end, and upsized threaded rod welded onto the other end.

Typically, the problems with de Burgh trusses include deterioration of the timber at the base of the verticals, corrosion and freezing of expansion bearings, loss of lateral alignment of the top chord, loss of shape of steel saddles under the verticals, and vibration damage to tension rods. Unlike earlier truss types, these trusses do not tend to experience brittle fracture of the tension rods as the tension rods in de Burgh and Dare trusses are made of steel rather than wrought iron.

Figure 139: Frozen Bearing at Lansdowne Bridge and Damaged Saddle Plates at Tabulam Bridge

For the purpose of determining section properties for spaced column assemblies, the same process as outlined in Section 4.6.3.4 may be applied to de Burgh trusses, as calculated in Table 17. Again, this table is not for determining the capacity of members, but only for determining equivalent values of the second moment of area to input into a global Microstran bridge model.

Table 17: Equivalent Second Moment of Area for Modelling de 91' & 104' Burgh Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Vertical Strut	Terminal Strut	Intermediate Strut
depth of flitches	100 mm	125 mm	100 mm
width of flitches	305 mm	305 mm	305 mm
length of assembly	3510 mm	3485 mm	3485 mm
I (Non-Composite)	$50x10^6mm^4$	$100x10^6mm^4$	$50x10^6mm^4$
N (Non-Composite)	640 kN	1300 kN	650 kN
N (Microstran Result)	1065 kN	1685 kN	1065 kN
I_x (Equivalent)	$85x10^{-6}m^4$	$130x10^{-6}m^4$	$80x10^{-6}m^4$
I_y ($=2bd^3/12$)	$475x10^{-6}m^4$	$590x10^{-6}m^4$	$475x10^{-6}m^4$

Although even the widest Allan trusses were constructed with timber cross girders, some of the wider de Burgh trusses incorporated metal fish-belly cross girders. The single lane de Burgh trusses, however, continued to make use of timber cross girders. It has been general practice to replace the timber cross girders (which are generally under-capacity) with steel hollow rectangular sections. Connections between the base of the timber vertical struts and the cross girders should be modelled as fixed rather than pinned in the global Microstran model of a de Burgh truss. This is because the detail of the connection is such that significant rotational restraint is provided.

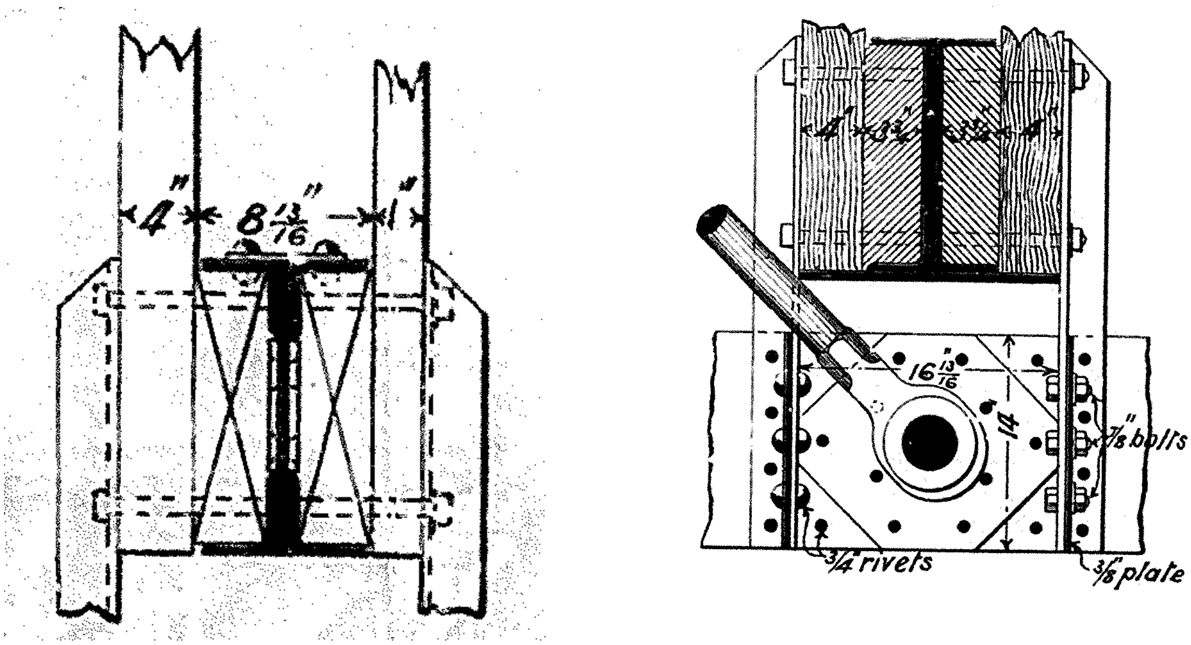


Figure 140: Details of Base Connection of Timber Struts with Bottom Chord and Metal Cross Girders

One end of the span is fixed in position and the other is free to move longitudinally. Both ends are pinned. Top chords and bottom chords are continuous. Tension rods are pinned at both ends. It is very important that tension rods be defined as tension-only members and vertical struts be defined as compression-only otherwise forces will be underestimated in some members. The end panels of the bottom chords may take compression or tension although only a three dimensional model will give compression forces because this is a three dimensional effect.

As is the case in an Allan Truss, so too in the de Burgh truss, the sway bracing connections have only a small fraction of the capacity required in order to provide effective lateral restraint to the top chord. Unless connections are upgraded, the top chord is unrestrained (see section 3.6.1).

The central panels in a de Burgh truss may experience stress reversals when the live load effects outweigh permanent effects. This is the reason for the counterbracing. It is particularly important that only realistic load cases are analysed using the non-linear analysis in Microstran. For example, live load should always be analysed in combination with dead load. If live load is analysed alone, excessive stress reversals will occur and the model will become unstable.

4.7.1 Modelling Timber Decks

The traditional decking system on Allan, de Burgh and Dare trusses consists of longitudinal stringers under transverse decking planks. In the 1950s and 1960s longitudinal sheeting was added to better distribute wheel loads and to provide a smoother running surface for vehicles. The longitudinal stringers extend over two panels lengths in a staggered pattern to ensure even distribution of loads to the cross girders. Sometimes the pattern of joints in the longitudinal stringers is changed over the life of the bridge. The locations of the joints will affect not only the bending moments in the stringers but also the bending moments in the cross girders, since the locations where stringers are continuous over the cross girders will tend to attract more load.

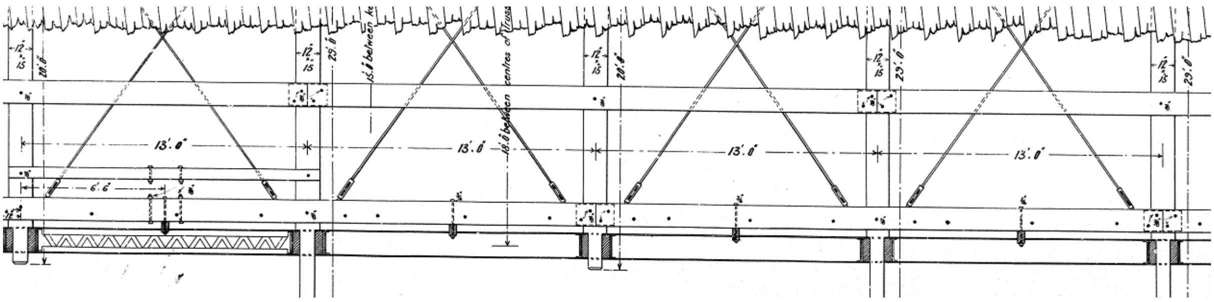


Figure 141: Layout of Timber Stringers and Transverse Decking on Typical de Burgh Truss Span

It is therefore important that analysis reflects the actual arrangement of longitudinal stringers when a traditional timber decking system is present on a timber truss span. It is also important that the analysis reflects the additional dead load present due to the longitudinal timber sheeting and spray seal, which are not shown on the original drawings, but are required for modern traffic.

The wearing surface generally consists of a spray seal, between 10 and 20 mm thick. Similarly, the timber kerbs do not add to the strength or stiffness of the bridge, but to the self weight.

Under the wearing surface there is generally longitudinal timber sheeting. This may be between 50 and 125 mm thick, and is closely spaced to allow application of the spray seal. Butt joints in longitudinal sheeting are staggered, and sheeting is bolted to the transverse decking at each end, and at approximately 1.5m centres along its length. Sheeting does not contribute to the structural capacity of the bridge, but does distribute wheel loads to the transverse decking. It is also possible that thicker longitudinal sheeting (100 - 125 mm) with tight butt joints may transmit longitudinal breaking forces by compression. The transverse decking is generally placed with gaps between elements for durability reasons, and these gaps are generally 20 to 50 mm wide.

It has been standard practice on many of these timber truss bridges to replace the traditional timber decking system with a stress laminated timber (SLT) deck. The SLT deck has many advantages. Although a traditional decking system may only last 10 years, and SLT deck, properly maintained, should last in excess of 50 years. The SLT deck is waterproof, and so it also improves the durability of the cross girders. Since it is still timber, it does not substantially increase the dead load of the span, and does not introduce any unwanted thermal effects.

4.7.2 Effects of Vibrations in de Burgh Trusses

Due to the configuration of de Burgh trusses, this truss type is particularly susceptible to vibration effects. Under the effects of a heavy vehicle crossing the bridge, the tension rods, in particular, have a tendency to wobble, sometimes with quite large deflections. If there is a bolt nearby then tension rods can get damaged due to the repeated impact against the bolt. This can occur at barrier post connections approximately midway down the tension rods, and also where the bolts for the tops of the verticals are sometimes located very close to the tension rods. It is important that bolts not be longer than necessary, so verticals should be clamped then bolted when installed, rather than achieving the bow in the verticals by tightening over-length top bolts.

Figure 142: Damage to de Burgh Truss Tension Rods due to Nearby Bolts and Vibration Effects.

4.8 Modelling Dare Trusses

Like the Allan trusses and de Burgh trusses, Dare trusses depend on relatively pure truss action and their structural behaviour is relatively easily to understand and model. Apart from the steel bottom chords and tension rods, the most significant structural difference between the Allan and the Dare truss is the fact that none of the timber flitches are curved in the Dare truss. This simplifies analysis as the timber is subject only to compressive forces rather than combined bending and compression, and it also simplifies the geometry, especially of the timber spacers.

Dare experimented a little with the cast iron shoe design, and so different configurations are used on different bridges. For example, at Bendemeer (Figure 143), the timber is cut to shape the shoe, whereas at Warroo (Figure 144), the shoe is shaped so that the timber is cut square. On most Dare trusses, there is a single bolt attaching the diagonal to the shoe (Figure 145), whereas on some (eg. Coonamit and Gee Gee) there are larger side plates on the shoes with two bolts.

Typically, the problems with Dare trusses include corrosion and freezing of expansion bearings and loss of lateral alignment of the timber top chord. Another issue which appears to be particularly prevalent in Dare trusses is separation and deterioration at the shear key connection between the diagonals and the top chord, some examples of which are given in Figures 143 - 145.

Figure 143: Deterioration at Top Chord Shoe in Dare Truss at Bendemeer

Figure 144: Deterioration at Top Chord Shoe in Dare Truss at Warroo

Figure 145: Deterioration at Top Chord Shoe in Dare Truss at Scabbing Flat

One reason why timber above Dare truss top shoes may be more susceptible to deterioration is that there is less drainage in Dare's standard details than in Allan's standard details. In Dare's top chord shoes, there is a round hole large enough for the tension rod to fit though but any water that comes down in the gap between the top chord flitches may tend to sit on the top of the shoe, and with very little drainage and no sunlight, this may cause the top chord to rot. This problem is most effectively dealt with by the application of protective flashing to the top chord.

Another reason could be overstress. For the 70' spans, both the Allan and the Dare truss have a square panel geometry 10' long and 10' high. For this 70' span geometry, Allan provided shear lugs of 1½" and 1" for both the first and second diagonals, while Dare increased the shear lug depths to 1¾" and 1¼" for the first diagonal, and reduced the shear lug depths to 1¼" and ¾" for the second diagonal, which appropriately reflects the lower stresses in the second diagonal.

However, when considering the 90' spans, Allan changed the shape of the panel to rectangular (10' long and 13' high), thereby reducing the horizontal component of the force from a diagonal acting on the top chord and increasing the number of diagonals taking the load, and he therefore continued to provide shear lugs of 1½" and 1" at both the first and second diagonal (with a single 1½" lug at the third). Dare, on the other hand, maintained the square truss geometry, so his panel sizes increased to 13' long by 13' high without any increase in the number of panels, but an increase in load. Despite this, he retained the shear lug depths of 1¾" and 1¼" for the first diagonal, and 1¼" and ¾" for the second diagonal, and even for the 104' spans, maintained the same details for the first diagonal and increased the second diagonal only to 1½" and 1¼".

In addition to this, in the Dare trusses, the shoes are held up with only one pair of coach screws located on the lighter side of the shoe, well away from the centre of gravity, meaning gaps do tend to open up between the top chord and the shoe, which reduces the depth of shear lug even more. It seems clear that additional connections are required to keep top chord shoes in place.

Figure 146: Separation of Shoes from Top Chord in Dare Truss over Williams River, Cooreei Bridge

First and second diagonals in Dare trusses are generally under strength, and this is sometimes seen on site with the these diagonals having a bowed shape, indicating overstress by buckling.

For the purpose of determining section properties for spaced column assemblies, the same process as outlined in Section 4.6.3.4 may be applied to Dare trusses, as calculated in Tables 18 - 20. Again, these tables are not for determining the capacity, but only for determining equivalent values of the second moment of area to input into a global Microstran bridge model.

Because dimensions vary from truss to truss, typical approximate dimensions are given below.

Table 18: Equivalent Second Moment of Area for Modelling Typical 70' Dare Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Brace G	Brace H	Brace K
depth of flitches	150 mm	115 mm	100 mm
width of flitches	305 mm	200 mm	180 mm
length of assembly	3760 mm	3610 mm	3610 mm
N (Microstran Result)	2210 kN	795 kN	545 kN
I_x (Equivalent)	$200\text{x}10^{-6}\text{m}^4$	$65\text{x}10^{-6}\text{m}^4$	$45\text{x}10^{-6}\text{m}^4$
I_y ($=2bd^3/12$)	$710\text{x}10^{-6}\text{m}^4$	$155\text{x}10^{-6}\text{m}^4$	$95\text{x}10^{-6}\text{m}^4$

Table 19: Equivalent Second Moment of Area for Modelling Typical 91' Dare Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Brace G	Brace H	Brace K
depth of flitches	150 mm	115 mm	115 mm
width of flitches	335 mm	230 mm	200 mm
length of assembly	4875 mm	4940 mm	4940 mm
N (Microstran Result)	1385 kN	525 kN	475 kN
I_x (Equivalent)	$210\text{x}10^{-6}\text{m}^4$	$80\text{x}10^{-6}\text{m}^4$	$75\text{x}10^{-6}\text{m}^4$
I_y ($=2bd^3/12$)	$900\text{x}10^{-6}\text{m}^4$	$230\text{x}10^{-6}\text{m}^4$	$150\text{x}10^{-6}\text{m}^4$

Table 20: Equivalent Second Moment of Area for Modelling Typical 104' Dare Truss Spaced Column

$$N = \frac{\pi^2 EI}{L^2}\ ;\ I = \frac{NL^2}{\pi^2 E}$$

	Brace J	Brace K	Brace L	Brace M
depth of flitches	180 mm	140 mm	115 mm	115 mm
width of flitches	355 mm	255 mm	230 mm	200 mm
length of assembly	4875 mm	4940 mm	4940 mm	4940 mm
N (Microstran Result)	2510 kN	940 kN	550 kN	500 kN
I_x (Equivalent)	$380\text{x}10^{-6}\text{m}^4$	$145\text{x}10^{-6}\text{m}^4$	$85\text{x}10^{-6}\text{m}^4$	$75\text{x}10^{-6}\text{m}^4$
I_y ($=2bd^3/12$)	$1340\text{x}10^{-6}\text{m}^4$	$385\text{x}10^{-6}\text{m}^4$	$235\text{x}10^{-6}\text{m}^4$	$155\text{x}10^{-6}\text{m}^4$

4.9 References

- Australian Hardwood & Cypress Technical & Detailing Guide
- Australian Standard AS 1720.1-2010, Timber Structures Part 1: Design Methods, 2010
- Boughton, G.N. & Crews, K.I. (1998), Timber Design Handbook in Accordance with the Australian Limit State Timber Design Code AS 1720, Australian Standards SAA HB108-1998
- Bootle, K.R. (1983), Wood in Australia, Types, Properties and Uses, Sydney: McGraw Hill Book Company Australia Pty Ltd, 1983
- Buchanan, A.H., Johns, K.C., Madsen, B (1985) "Column Design Methods for Timber Engineering" International Council for Building Research Studies and Documentation – Working Commission W18 – Timber Structures, Meeting 18, Israel
- CSIRO (1948) "Handbook of Structural Timber Design – Third Edition", J.J. Gourley, Government Printer, Melbourne.
- Department of Main Roads, New South Wales (1962), Manual No. 6, Bridge Maintenance.
- Department of Main Roads, New South Wales (1987), The Aesthetics of Bridges.
- Federal Highway Administration (2005), Covered Bridge Manual, Publication no. FHWA_HRT-04-098, U.S. Department of Transportation, April 2005
- Leicester, R.H. (1986), Creep Buckling Strength of Timber Beams and Columns, International Council for Building Research Studies and Documentation, Working Commission W18, Timber Structures, Meeting 19, Italy, 1986
- Leicester, R.H., (1988) "Format for Buckling Strength", International Council for Building Research Studies and Documentation – Working Commission W18 – Timber Structures, Meeting 21, Canada
- Madson, B. (2000), Behaviour of Timber Connections, Canada: Timber Engineering Ltd
- Madson, B. (1992), Structural Behaviour of Timber, Canada: Timber Engineering Ltd, 1992
- NCHRP (2005), A Context for Common Historic Bridge Types, NCHRP Project 25-25 Task 15, prepared for the National Cooperative Highway Research Program, October 2005
- Nicholas, A. (2011), Capacity of Compression Members in RTA Timber Truss Bridges, 2011
- Nordic Innovation (2002), Nordic Wood Special, Nordic Industrial Fund, 2002
- Roads and Maritime Services (2006), RMS QC Specification 2380: Timber for Bridges
- Roads and Maritime Services (2008), RMS QC Specification M757: Timber Truss Repairs
- Smith, L. (2008), Use of Australian Hardwoods in Heavy Structures in Southern New Zealand, *Australian Journal of Multi-disciplinary Engineering*, Institution of Engineers Australia Vol 6 No 1, 2008, pp 77-86
- Wilkinson, K. (2008), Capacity Evaluation and Retrofitting of Timber Bridge Girders, PhD Thesis, Queensland University of Technology.
- Ximenes, F. et al (2006), Forests, Wood and Australia's Carbon Balance, Forest & Wood Products Australia and Cooperative Research Centre for Greenhouse Accounting, 2006
- Yttrup, P.J, Law, P.W. & Audova, H. (1991), The Mechanics of Timber Beam bridge, Bridges, Part of the Transport System, AUSTROADS Conference, Brisbane 1991

5 Design and Assessment of Timber Bridges

5.1 Design Loads

Design loadings for timber bridges shall comply with AS 5100.2-2017, excluding Section 6 (dead loads), Section 7 (road traffic) and Section 10 (minimum restraint load).

Minimum additional design loadings for timber bridges shall comply with Sections 5.1.1 and 5.1.2 below. Load factors for these additional design loadings are given in Table 21 below.

5.1.1 Dead Loads

The minimum dead load per unit volume of any timber component (excluding kiln-dried timber) shall be taken as 11 kN/m^3. The minimum dead load per unit volume for kiln-dried timber (as used in stress laminated timber (SLT) decks) shall also be taken as 11 kN/m^3.

The design loads for serviceability and ultimate limit states shall be obtained by applying the appropriate load factor in Table 21 to the nominal loads on the structure. Where the dead load is calculated from the dimensions shown on the drawings, the "design case" load factor applies. Where an assessment of an existing member is being undertaken, and dead load is calculated from actual dimensions measured on site, the "direct measurement" load factor applies.

Whether dead load is calculated from drawings or from site measurements, care shall be taken to ensure that all relevant elements are included in dead load calculations. For example, longitudinal timber sheeting varying in depth from 50 mm to 100 mm is provided on almost all timber bridges despite not being present in original drawings. Also, on traditional timber decks consisting of transverse decking and longitudinal sheeting, a spray seal is provided. The dead load from the sheeting, and the superimposed dead load from the spray seal must be calculated.

Care shall be taken to ensure that all metal components (such as cast iron or welded steel shoes splice plates, monorails with monorail attachments, utilities with attachments and barriers) are included in dead load calculations. A load factor of 1.1 shall be applied to metal components.

Table 21: Load Factors for Timber Bridge Design and Assessment

Type of Load	Serviceability Limit State	Ultimate Limit States	
		Load Reduces Safety	Load Increases Safety
Dead Load (design case)	1.0	1.4	0.8
Dead Load (direct measurement)	1.0	1.2	0.9
Dead Load (SLT Deck)	1.0	1.2	0.9
Superimposed Dead Load	1.0	2.0	0.0
W7 Wheel Load + DLA	1.0	2.0	N/A
T44 Truck Load + DLA	1.0	2.0	N/A
Braking Force	1.0	1.8	N/A

5.1.2 Live Loads

Unless the bridge is to have a posted and enforced load limitation, design live loads shall comply with the following clauses of AS 5100.7-2004 Appendix A, with the load factors in Table 21:

- A2.2.2 T44 Truck Loading;
- A2.2.5 Number of Lanes for Design and Lateral Position of Loads;
- A2.2.6 Modification Factors for Multiple Lane Bridges; and
- A2.2.7 Design for Localised Load Effects – W7 Wheel Loading.

5.1.2.1 Dynamic Load Allowance

The Dynamic Load Allowance (DLA) for timber bridges shall not be less than 0.2 irrespective of the expected vehicle speed. The DLA applies to both the ultimate and serviceability limit states.

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

For bridges with SLT decks, the Dynamic Load Allowance shall not be less than 0.25.

5.1.2.2 Braking Force

Braking effects shall be considered as a longitudinal force acting at deck surface level, and shall be applied in either direction. Irrespective of the width of the structure or the speed of the vehicles, the nominal longitudinal force shall not be less than 200 kN, with the load factors in Table 21.

This nominal braking force of 200kN is the minimum requirement of AS5100-2017, and is approximately equivalent to a T44 truck braking with a deceleration of 0.45g. Testing in Australia has shown that for general driving conditions in a 60km/hr speed zone, trucks decelerate at approximately 0.3g (stopping distance of 47m), but in urgent situations trucks have been shown to achieve decelerations up to 0.75g (stopping distance of 19m). The Australian design rules require braking systems to be capable of decelerating heavy vehicles at a minimum rate of approximately 0.45g. The ultimate limit state braking forces that have been adopted correspond to a range of mass and deceleration rates that are considered reasonable for the expected traffic conditions for timber bridges on New South Wales roads.

5.1.2.3 Minimum Lateral Restraint

To ensure that the superstructure has sufficient lateral restraint to resist lateral forces not otherwise allowed for in the design, a positive lateral restraint system between the superstructure and the substructure shall be provided at abutments and piers. The restraint system shall be capable of resisting a minimum ultimate design horizontal force perpendicular to the bridge centreline of 200 kN at each abutment and pier, which need not be loaded concurrently.

A load factor of 1.0 shall be used.

5.2 Design Parameters

5.2.1 Capacity Reduction Factors

Values of the capacity factor (ϕ) for calculating the design capacity of primary and secondary structural members and structural connections are listed in Table 22. Primary structural elements are members and connections whose failure could result in collapse of a significant portion of a structure. These include timber girders, all truss elements and elements in piers. Secondary structural elements include members and connections whose failure would result in a localised collapse. These include stringers (spaning between cross girders) and decking elements.

Table 22: Capacity Reduction Factors (ϕ)

	secondary structural element	primary structural element
Sawn timber F-grade F17 and higher	0.85	0.75
Round timber	0.70	0.60
Bolts or coach screws M16 and smaller	0.80	0.75
Bolts or coach screws larger than M16	0.65	0.60

5.2.2 Modification Factors

5.2.2.1 Duration of Load Factor (k_1)

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

- $k_1 = 0.57$ for permanent actions e.g., dead load, superimposed dead load, earth pressure
- $k_1 = 0.80$ for serviceability live load
- $k_1 = 0.97$ for ultimate design live load (T44)
- $k_1 = 0.94$ for ultimate assessment live load where vehicle is less than T44 (eg, ST 42.5)
- $k_1 = 1.00$ for other ultimate actions e.g., braking force, minimum lateral restraint, log impact

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

- $k_1 = 0.57$ for permanent actions e.g., dead load, superimposed dead load, earth pressure
- $k_1 = 0.69$ for serviceability live load
- $k_1 = 0.86$ for ultimate live load (T44)
- $k_1 = 0.77$ for ultimate assessment live load where vehicle is less than T44 (eg, ST 42.5)
- $k_1 = 1.00$ for other ultimate actions e.g., braking force, minimum lateral restraint, log impact

Note that for any given combination of loads of differing duration, the factor k_1 to be used is that appropriate to the action that is of the shortest duration. For example, when considering ultimate dead load plus ultimate live load (T44), the appropriate member k_1 factor is 0.97.

5.2.2.2 Size Factor (k_{11})

The capacity in bending and tension for sawn timber shall be modified by the size factor k_{11}:

- For bending k_{11} shall be the lesser of $(300/d)^{0.167}$ or 1.
- For tension parallel to grain, k_{11} shall be the lesser of $(150/d)^{0.167}$ or 1.

5.2.2.3 Factors for Bolted Connections (k_{16} and k_{17})

Thick steel side plates can increase the capacity of a bolt in bearing on the timber by restricting bolt rotation within the members. In order for this to be effective, the plate must be thick and stiff enough to give effective bending restraint to the bolt. k_{16} shall generally be taken as 1.0.

k_{16} may be taken as 1.2 for bolts that transfer load through two metal side plates, one on each side of the timber, only where the bolts are a close fit to the holes in these plates, and where metal plates are of adequate strength and stiffness to induce double curvature in the bolt. It would be very rare for these requirements to be met, and so k_{16} should generally be taken as 1.0.

When using unseasoned timber, consideration must be given to the effects of shrinkage especially when detailing connections. 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 the grain.

Restraint to timber shrinkage due to the detailing of bolted connections in unseasoned timber causes splitting of timber with a loss of capacity equivalent to specifying half the number of bolts. In addition to the loss of capacity, there is a reduction in durability of the timber as premature splitting allows moisture ingress. Connections shall therefore be detailed, wherever possible to ensure no restraint to timber shrinkage. Examples of poor detailing are given below.

Where timber shrinkage is restrained by a bolted connection, a k_{17} factor of 0.5 shall apply.

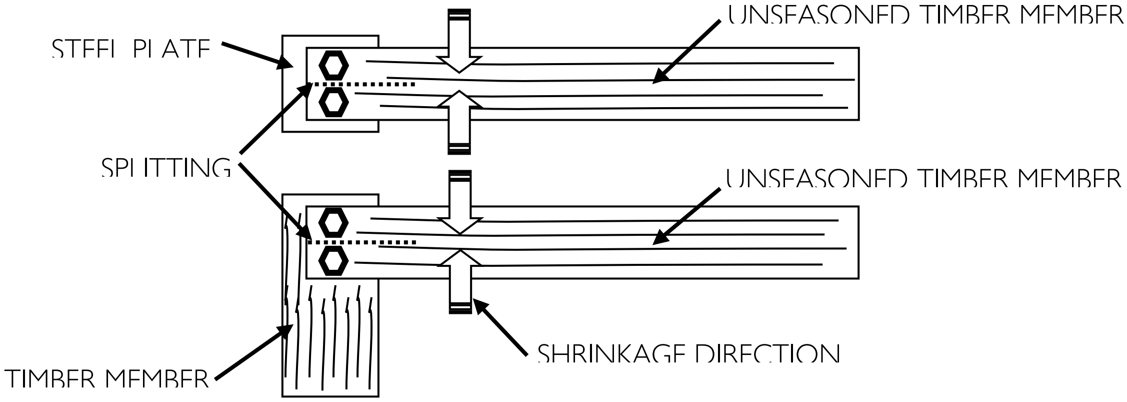


Figure 147: Poor Detailing: Timber shrinkage restrained by steel plate and longitudinal grain

5.2.2.4 Shaving Factor (k_{21})

Where round timbers (such as in pier trestles or girders) are shaved on one or more faces, the modulus of elasticity shall be reduced by 5% and a shaving factor k_{21} shall be applied.

Table 23: Shaving Factor for Round Timber (k_{21})	
Shaved all faces, subject to bending	0.85
Shaved on compression face only, subject to bending	0.95
Shaved on all or any face, subject to compression parallel to grain	0.95
Shaved on all or any face, subject to compression perpendicular to grain or shear	1.00
Shaved on all or any face, subject to tension	0.85

5.2.3 Characteristic Values

The characteristic strength and stiffness properties for the design of structural timber elements shall be taken from Table 24 below. When determining the appropriate stress grade, reference shall be made to RMS QC Specification 2380, Table 2380/1, Strength and Durability Requirements. In the absence of information to the contrary, the minimum stress grade given in RMS 2380, Table 2380/1 shall be used for design and assessment purposes.

Table 24: Characteristic Values for Timber Design and Assessment (MPa)

Stress Grade	Bending (f'_b)	Tension parallel to grain (f'_t)	Shear (f'_s)	Compression parallel to grain (f'_c)	Modulus of Elasticity (E)
F27	67	42	5.1	51	18 500
F22	55	34	4.2	42	16 000
F17	42	25	3.6	34	14 000

5.2.3.1 Modulus of Elasticity

The values for modulus of elasticity given in Table 24 are average values including an allowance of about 5 percent for shear deformation. These values are appropriate for use in global models. For estimating the lower and higher fifth percentile values (necessary when determining forces due to permanent deflections or buckling capacity), expressions are given here:

- $E_{0.05} = 0.5E_{average}$
- $E_{0.95} = 1.5E_{average}$

Modulus of elasticity varies considerably between pieces of timber, even of the same species and stress grade. The modulus of elasticity also varies along the length of a piece of timber. In addition to this, the modulus of elasticity varies with changes in moisture content, and also varies according to the age of the timber, with deterioration causing a reduction in stiffness with time.

5.3 Durability

The structure and its structural elements (including timber, metal, adhesives and other structural material) shall be designed to satisfy the strength, stability and serviceability requirements for the design life of the structure. Any assumed maintenance program shall be specified in the design drawings. Due consideration shall be given to the local environmental conditions, such as the humidity and temperature as well as any biological agents (eg. termites) that may act on the structure to reduce its performance characteristics. The water falling upon the structure shall be minimised as much as is practical; the structure shall be designed so that water drains away as rapidly as possible; the structure shall be designed so that water and dirt do not become entrapped; and all parts of the structure shall be designed to be well ventilated as much as is practical. Consideration shall be given to the following strategies for improved durability:

- Limit moisture on the timber by partial or complete covering of the main structural elements, carefully designed to limit moisture due to condensation (ie. flashing);
- Limit standing water on timber through appropriate inclination of surfaces;
- Limit openings, slots, etc. where water may accumulate or infiltrate;
- Limit direct absorption of water (e.g. capillary absorption from concrete or from ground) through use of appropriate barriers, and by keeping elements clear of ground and vegetation;
- Limit extended areas of timber steel interfaces;
- Limit exposure of timber end grain by appropriate sealing and/or cover plates;
- Limit swelling and shrinkage movements by use of timber seasoned to close to the equilibrium moisture content for the environment in which the timber is to be used;
- Design connections to maximise natural ventilation of all timber parts; and
- Specify application and maintenance of appropriate protective paint system.

Proper flashing and / or protective paint systems are essential to timber truss components. Any damaged or poorly maintained protection system can cause more harm than good. Paint systems have an initial benefit of delaying the onset of moisture entry and hence of decay. However, once cracks occur, moisture enters the timber and low quality or poorly maintained systems may inhibit drying and therefore accelerate the progress of decay. The rate of decay is also accelerated if the coating is a dark colour, because this encourages the wood to absorb the heat from the sun, so light coloured finishes are preferred. The application and maintenance of a light coloured quality acrylic paint system will extend the service life of a naturally durable timber.

Timber is almost always somewhat acidic, and the acidic constituents in timber are likely to be more reactive when the timber becomes wetted or if it is unseasoned (both of which occur in timber bridges), at which time timber may also contain a small amount of acetic acid. Steel's rate of corrosion when in contact with damp timber increases very considerably. Furthermore, when corrosion of the metal occurs, timber in contact with the metal gradually loses much of its tensile strength, because rusting metal catalyses the oxidation of the polysaccharides of the timber.

5.4 Design Capacity

5.4.1 Members Subject to Bending

5.4.1.1 Bending Capacity

For the design of members subject to bending, which are not subject to slenderness effects, the provisions of this Section shall be satisfied. This includes members of round or square cross section, members subject to bending about their minor axis only, and members subject to bending about their major axis which have continuous lateral restraint to their compression edge.

$M_d \geq M^*$ where $M_d = \phi\, k_1\, k_{11}\, k_{21}\, f'_b\, Z$

M_d	=	design capacity in bending
M^*	=	design action effect in bending
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
k_{11}	=	size factor
	=	in accordance with Section 5.2.2.2 for rectangular sections
	=	1.0 for round sections
k_{21}	=	shaving factor
	=	1.0 for rectangular sections
	=	in accordance with Table 23, Section 5.2.2.4 for round sections
f'_b	=	characteristic value in bending (see Table 24, Section 5.2.3)
Z	=	section modulus
	=	$db^2/6$ for bending of rectangular sections about minor axis
	=	$bd^2/6$ for bending of rectangular sections about major axis
	=	$\pi d^3/32$ for round sections

For elements of a truss consisting of an assembly of two members with permanent bending due to fabrication (eg, verticals in a de Burgh truss and diagonals in an Allan truss), care shall be taken to include these permanent effects as well as secondary bending moments due to eccentricity of axial forces. The bending moment due to fabrication is calculated as follows:

$$M_{fabrication} = \frac{48EI\delta}{4L^2}$$

For such members, bending moments due to fabrication are calculated with three values of E relating to the lower 5th percentile, the highest 95th percentile, and the characteristic average. These values of the modulus of elasticity E are determined in accordance with Section 5.2.3.1.

For assessment of existing members, or for the design of new members that shall be fabricated a minimum of 12 months prior to installation into the bridge, a reduction factor of 0.5 may be applied to the design bending moment due to fabrication to account for stress relaxation.

5.4.1.2 Flexural Shear Capacity

For the design of members subject to shear, the provisions of this Section shall be satisfied. In calculating the design action effect in shear, it is appropriate to disregard the design actions located within a distance of 1.5 times the depth of the beam from the inside face of the support.

$V_d \geq V^*$ where $V_d = \phi\, k_1\, f'_s\, A_s$

V_d	=	design capacity in shear
V^*	=	design action effect in shear
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
f'_s	=	characteristic value in shear (see Table 24, Section 5.2.3)
A_s	=	shear plane area
	=	2/3 (bd) for rectangular sections
	=	$3\pi d^2/16$ for round sections

5.4.1.3 Bearing Capacity

The design capacity in bearing perpendicular to the grain shall satisfy the following:

$N_{d,p} \geq N_p^*$ where $N_{d,p} = \phi\, k_1\, f'_p\, A_p$

$N_{d,p}$	=	design capacity in bearing perpendicular to the grain
N_p^*	=	design action effect in bearing perpendicular to the grain
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
f'_p	=	characteristic value in bearing perpendicular to the grain (see Table 25)
A_p	=	bearing area for loading perpendicular to the grain

The design capacity in bearing parallel to the grain shall satisfy the following:

$N_{d,l} \geq N_l^*$ where $N_{d,l} = \phi\, k_1\, f'_l\, A_l$

$N_{d,l}$	=	design capacity in bearing parallel to the grain
N_l^*	=	design action effect in bearing parallel to the grain
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
f'_l	=	characteristic value in bearing parallel to the grain (see Table 25)
A_l	=	bearing area for loading parallel to the grain

Table 25: Characteristic Values for Bearing Perpendicular and Parallel to Grain (MPa)

Strength Group (unseasoned)	Perpendicular to Grain - f'_p	Parallel to Grain - f'_l
S1	17	51
S2	13	40
S3	10	30

5.4.2 Members Subject to Axial Forces

5.4.2.1 Compression Capacity

For design of members subject to compression, the provisions of this Section shall be satisfied.

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\, k_1\, f'_c\, A_c$; or
$= \phi\, N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
f'_c	=	characteristic value, compression parallel to grain (Table 24, Section 5.2.3)
A_c	=	cross-sectional area
N_{cr}	=	critical elastic buckling load of the member (see below)

The critical elastic buckling load N_{cr} of a member shall be determined as follows:

$$N_{cr} = \frac{\pi^2 E_{0.05} I}{(g_{13} L)^2}$$

where

$E_{0.05}$	=	lower 5th percentile modulus of elasticity (Section 5.2.3.1)
I	=	second moment of area about the minor axis
	=	$db^3/12$ for a rectangular section
	=	$\pi d^4/64$ for round sections
g_{13}	=	effective length factor (see Table 26 below)
L	=	length of member in mm

Alternatively, the critical elastic buckling load N_{cr} of a member or assembly shall be determined by a Microstran model prepared in accordance with Section 4.6.3.4 but with the lower 5th percentile value of the modulus of elasticity used rather than the average or characteristic value. Although, when using an elastic critical buckling analysis to determine equivalent section properties, it is important that both ends are be modelled as pinned in all cases, when capacities (N_{cr}) are being determined, for Allan, de Burgh and Dare trusses, the bases shall be fixed.

Table 26: Effective Length Factor g_{13} for Compression Members

Condition of End Restraint	g_{13}
Restrained at both ends in position and direction (Allan, de Burgh and Dare trusses)	0.70
One end fixed in position and direction, other end restrained in position only	0.85
Restrained in position only (Old PWD and McDonald trusses, excluding principals)	1.00
Restrained in position and direction at one end and free at the other end	2.00

5.4.2.2 Force on Lateral Restraints

In computing the load capacity of a compression member of length (L) with n intermediate lateral restraints, the design force (N^*_R) on each lateral restraint is estimated as follows:

$$N^*_R = \frac{0.1N^*_c}{n+1} h_{26} h_{27} g_{38}$$

where

N^*_c	=	design action effect in compression
n	=	number of equally spaced intermediate restraints
h_{26}	=	2.0 when loads are dead loads only and timber is initially unseasoned
	=	1.5 when loads are dead plus live loads and timber is initially unseasoned
h_{27}	=	1.0 for sawn timbers
g_{38}	=	lesser of (m + 1)/2 and 5
m	=	number of members supported by restraint system

When sway bracing providing lateral restraint to a top chord in a truss span is being assessed using this formula, the component of the force in the direction of the sway bracing must be calculated (the formula above gives the horizontal component only). In addition to this, the vertical component from the diagonal sway bracing must be calculated in order to assess the capacity of the connections between the sway bracing and the top chord.

5.4.2.3 Tension Capacity

For the design of members subject to tension, the provisions of this Section shall be satisfied.

$N_{d,t} \geq N^*_t$ where $N_{d,t} = \phi\, k_1\, k_{11}\, k_{21}\, f'_t\, A_t$

$N_{d,t}$	=	design capacity in tension
N^*_t	=	design action effect in tension
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
k_{11}	=	size factor
	=	in accordance with Section 5.2.2.2 for rectangular sections
	=	1.0 for round sections
k_{21}	=	shaving factor
	=	1.0 for rectangular sections
	=	in accordance with Table 23, Section 5.2.2.4 for round sections
f'_t	=	characteristic value in tension parallel to grain (Table 24, Section 5.2.3)
A_t	=	net cross-sectional area of tension member

5.4.3 Members Subject to Combined Actions

5.4.3.1 Combined Bending and Compression

In order to calculate the capacity of a timber member subject to combined compression and bending, the following simplification of the biaxial bending formula shall be satisfied:

$$\left(\frac{M^*_y}{M_{d,y}}\right) + \left(\frac{N^*_c}{N_{d,c}}\right) \leq 1.0$$

For elements of a truss consisting of an assembly of two members with permanent bending due to fabrication (eg, de Burgh truss verticals or Allan truss diagonals), this formula shall be satisfied for three values of the modulus of elasticity relating to the lower 5th percentile, the highest 95th percentile, and the characteristic average. These values of E determined in accordance with Section 5.2.3.1 shall be applied to bending stress and compressive strength calculations.

5.4.3.2 Combined Bending and Tension

For the design of sawn timber members subject to combined bending and tension, which are not subject to slenderness effects, the following formula shall be satisfied:

$$\left(\frac{M^*}{M_d}\right) + \left(\frac{N^*_t}{N_{d,t}}\right) \leq 1.0$$

5.4.4 Bolted Connections

For the purpose of connection design, connections are classified into two types as follows:

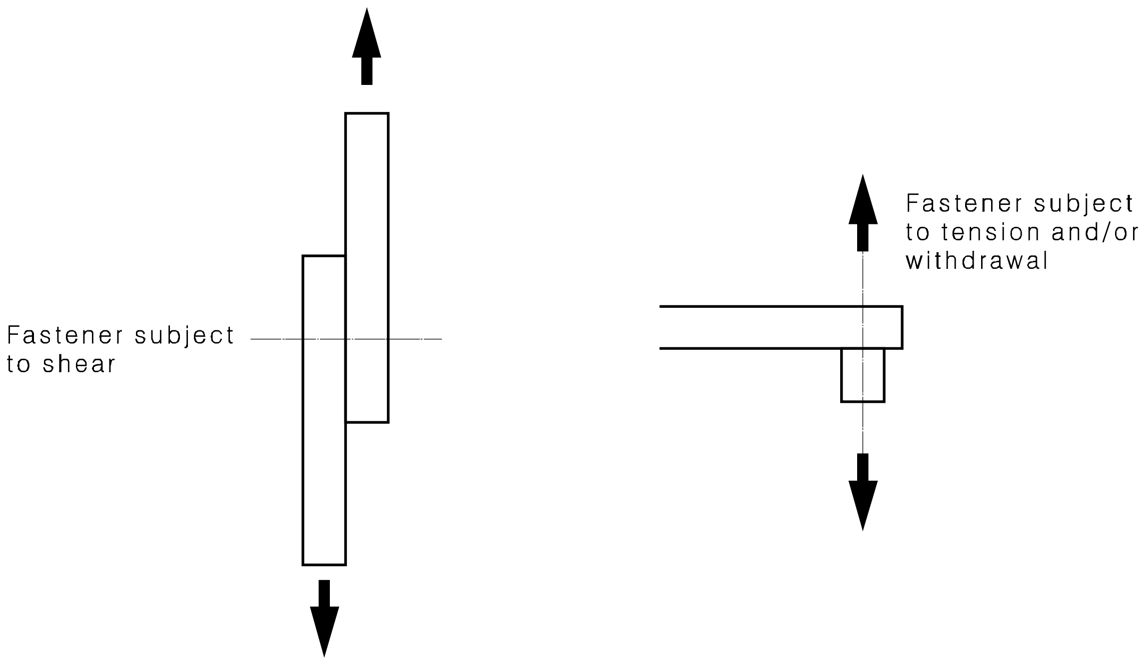


Figure 148: Left: Fastener subject to shear (Type 1); Right: Fastener subject to Tension (Type 2)

5.4.4.1 Type 1 Connections – Bolts Subject to Shear

For the design of bolted connections subject to shear, the provisions of this Section shall be satisfied. The provisions of this Section are applicable to bolts fitted into pre-bored holes of diameter approximately 10% greater than the bolt diameter and fitted with washers at each end.

$N_{d,j} \geq N^*_j$ where $N_{d,j} = \phi\, k_1\, k_{16}\, k_{17}\, n_b\, n_s\, Q$

$N_{d,j}$	=	design capacity in shear
N^*_j	=	design action effect in shear
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
k_{16}	=	generally 1.0 (see section 5.2.2.3)
k_{17}	=	factor for multiple bolted joint (see section 5.2.2.3)
n_b	=	number of bolts resisting design action effect in shear
n_s	=	number of shear planes through which bolt passes
Q	=	characteristic capacity of bolt
	=	see Tables 27 & 29 for bolts loaded perpendicular to grain
	=	see Tables 28 & 29 for bolts loaded parallel to grain

Table 27: Characteristic Capacity for Bolts Loaded Perpendicular to the Grain (Q) – Unseasoned Timber

Joint group	Timber thickness	M12	M16	M20	M24
J1	50	6 600	8 800	11 000	13 200
	75	9 150	13 200	16 500	19 800
	100	9 150	14 080	19 680	25 870
	150	9 150	14 080	19 680	25 870
	200	9 150	14 080	19 680	25 870
J2	50	5 250	7 000	8 750	10 500
	75	7 880	10 500	13 130	15 750
	100	8 730	13 440	17 500	21 000
	150	8 730	13 440	18 780	24 690
	200	8 730	13 440	18 780	24 690
J3	50	3 300	4 400	5 500	6 600
	75	4 950	6 600	8 250	9 900
	100	6 600	8 800	11 000	13 200
	150	6 860	10 560	14 760	19 400
	200	6 860	10 560	14 760	19 400

Table 28: Characteristic Capacity for Bolts Loaded Parallel to the Grain (Q) – Unseasoned Timber					
Joint group	Timber thickness	M12	M16	M20	M24
J1	50	13 200	22 200	27 800	33 300
	75	13 200	23 400	36 600	50 000
	100	13 200	23 400	36 600	52 700
	150	13 200	23 400	36 600	52 700
	200	13 200	23 400	36 600	52 700
J2	50	11 100	17 600	22 000	26 400
	75	11 100	19 700	30 800	39 600
	100	11 100	19 700	30 800	44 400
	150	11 100	19 700	30 800	44 400
	200	11 100	19 700	30 800	44 400
J3	50	10 200	14 200	17 800	21 300
	75	10 200	18 200	26 600	32 000
	100	10 200	18 200	28 400	40 900
	150	10 200	18 200	28 400	40 900
	200	10 200	18 200	28 400	40 900

5.4.4.2 Type 2 Connections – Bolts Subject to Tension

For the design of bolted connections subject to tension, the provisions of this Section shall be satisfied. The provisions of this Section are applicable to bolts fitted into pre-bored holes of diameter approximately 10% greater than the bolt diameter and fitted with washers at each end. In addition to checking the tensile capacity of the bolt in accordance with AS5100.6-2017, the possibility of crushing of the timber under the washer shall be checked as follows:

$N_{d,j} \geq N^*_j$ where $N_{d,j} = \phi\, k_1\, n_b\, f'_{pj}\, A_w$

$N_{d,j}$	=	design capacity in tension
N^*_j	=	design action effect in tension
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
n_b	=	number of bolts resisting design action effect in tension
f'_{pj}	=	characteristic bearing capacity for timber in joints
	=	22.0 for Joint Group 1 (see Table 29)
	=	17.5 for Joint Group 2 (see Table 29)
	=	11.0 for Joint Group 3 (see Table 29)
A_w	=	effective area of washer for bearing = 1/3 x area of washer

Strength group, joint group and durability class for common timber truss species are given below.

Table 29: Strength Group, Joint Group and Durability Class for Various Species

Common Name	Botanical Names	Strength Group	Natural in-ground durability class	Joint Group
Brown Bloodwood	Corymbia trachyphloia	S3	1	J2
Coast Grey Box or Bosisto's Box	Eucalyptus bosistoana	S1	1	J2
Coastal Blackbutt (free of heart)	Eucalyptus pilularis	S2	2	J2
Forest Red Gum	Eucalyptus tereticornis	S3	1	J2
Grey Box or Gum-Topped Box	Eucalyptus microcarpa Eucalyptus hemiphloia Eucalyptus woolsiana Eucalyptus moluccana	S2	1	J1
Grey Gum	Eucalyptus propinqua Eucalyptus punctata	S1	1	J2
Grey Ironbark	Eucalyptus paniculata Eucalyptus siderophloia	S1	1	J1
Gympie Messmate	Eucalyptus cloeziana	S2	1	J2
Red Bloodwood	Corymbia gummifera	S3	1	J2
Red Ironbark (Broad-Leaved)	Eucalyptus fibrosa	S1	1	J2
Red Ironbark (Narrow-Leaved)	Eucalyptus crebra	S2	1	J1
Red Ironbark	Eucalyptus sideroxylon	S2	1	J2
Red Mahogany	Eucalyptus resinifera	S2	2	J1
Spotted Gum	Corymbia maculata Corymbia citriodora Corymbia henryi	S2	2	J2
Steel Box	Eucalyptus rummeryi	S2	1	J2
Tallowwood	Eucalyptus microcorys	S2	1	J1
Turpentine	Syncarpia glomulifera	S3	1	J2
White Mahogany	Eucalyptus acmenoides	S2	1	J2
White Stringybark	Eucalyptus eugenioides	S3	3	J2
White Stringybark	Eucalyptus globoidea	S3	2	J2
White Topped Box	Eucalyptus quadrangulata	S2	2	J2
Woollybutt	Eucalyptus longifolia	S2	1	J2
Yellow Box	Eucalyptus melliodora	S3	1	J2
Yellow Stringybark	Eucalyptus muellerana	S3	3	J2

5.4.5 Shear Key Connections

Critical shear key connections occur in the bottom chords of Allan trusses as well as in the butting blocks of Old PWD and McDonald trusses. In Allan trusses, the shear keys consist of metal blocks riveted to metal splice plates against which the timber bears. In Old PWD and McDonald trusses, the shear keys are timber to timber connections. Shear key connections also occur in Allan and Dare truss shoes. In addition to designing the metal component to avoid structural failure, two aspects of timber need to be considered when designing or assessing shear key connections. These aspects are timber in bearing parallel to grain and timber in direct shear.

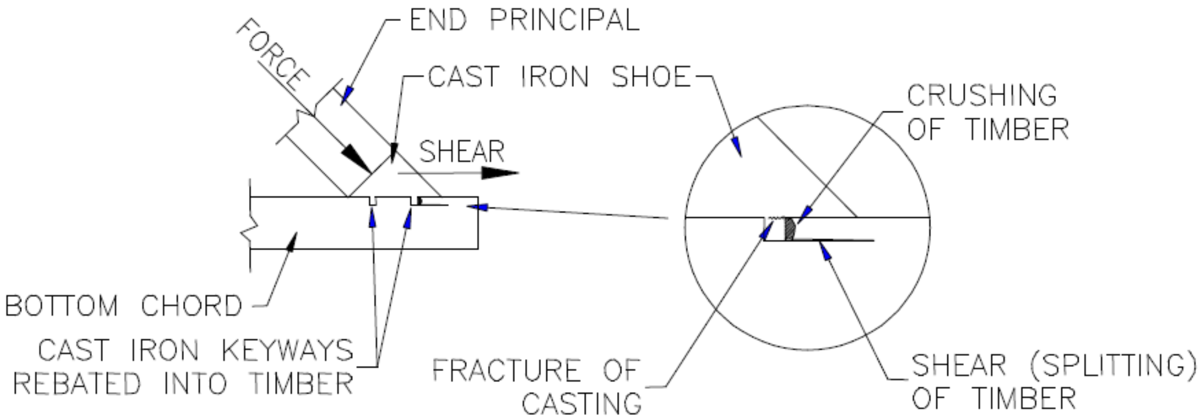


Figure 149: Possible Modes of Failure for Shear Key Type Connections in Timber

Bearing parallel to grain is checked in accordance with the provisions of Section 5.4.1.3.

The design capacity in direct shear along the grain shall satisfy the following:

$V_{d,j} \geq V^*$ where $V_{d,j} = \phi\, k_1\, f'_{ds}\, A_{ds}$

V_d	=	design capacity in direct shear
V^*	=	design action effect in direct shear
ϕ	=	capacity factor (see Table 22, Section 5.2.1)
k_1	=	duration of load factor (see Section 5.2.2.1)
f'_{ds}	=	characteristic value in direct shear (see Table 30)
A_{ds}	=	shear plane area = width of member x length of shear plane where the length of the shear plane is equal to the actual length of shear plane up to a maximum of 10 x shear key depth (eg, for 25 mm shear key, maximum shear plane length is 250 mm)

Table 30: Characteristic Values for Direct Shear Along the Grain (MPa)

Strength Group (unseasoned)	Parallel to Grain - f'_{ds}
S1	14.0
S2	11.5
S3	11.0

5.5 Design of Traffic Barriers on Timber Bridges

5.5.1 Understanding Traditional Timber Ordnance Fences

The early timber truss bridges had timber posts and railings consisting only of a top rail and a mid-rail connected to timber posts spaced approximately every 2m. On Old PWD trusses, the rails tended to finish at the abutments, but for McDonald trusses and later, some form of timber rail continued for road embankment approaches where warranted. On Old PWD trusses, kerbs were sometimes, but not always provided on approach spans, but for McDonald trusses and later, timber kerbs were always provided on both truss spans and approach spans. The purpose of the barriers was purely to provide visual delineation (especially to horses, bullocks, sheep and cows, who were the most frequent bridge users), and for this reason they were white. They had sufficient strength for a pedestrian to gently lean on, but zero resistance to impact loads.

Figure 150: Typical Failure of Timber Barriers Under Impact Loading

It was not until the turn of the century when the de Burgh trusses were being built that wire was detailed in the drawings as a standard component of the timber rails for both truss and approach spans. The galvanised steel wires (Fig 151), which extended the length of the bridge, were usually located between the top and mid rail, and also between the mid rail and the kerb. The wires, although only 3 mm in diameter and hardly visible, provide some (small) resistance to impact loads. This is due to the fact that they are structurally continuous for the length of the bridge and held solidly in place by being threaded through holes in the centres of the timber barrier posts.

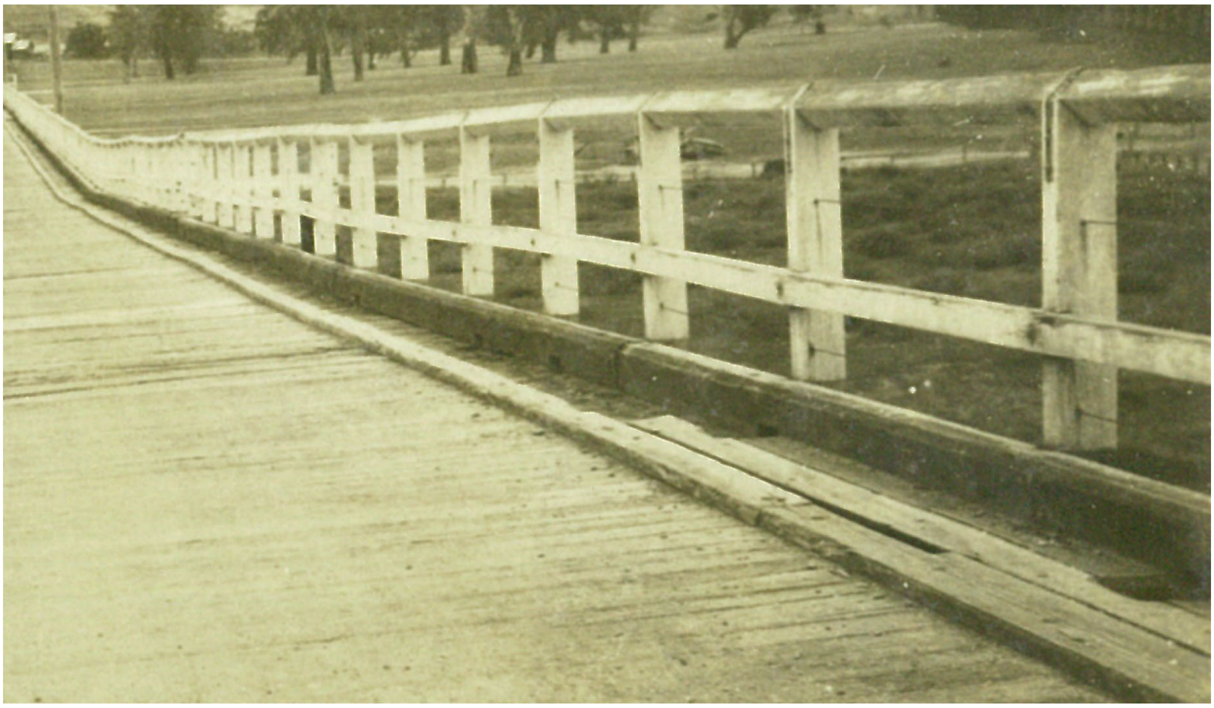


Figure 151: Prince Alfred Bridge Viaduct (1929) – Photograph Showing Timber Rails with Steel Wire

The report below shows that travel was a dangerous business when timber truss bridges were being constructed, and also shows that the steel wires were, at least sometimes, effective.

> *"About 8 o'clock on Wednesday morning a serious accident occurred on the Prince Alfred Bridge, Gundagai. A team of horses with a load of corn weighing about 5½ tons, in charge of Charles Field, was crossing the bridge going in the direction of the railway station, when it came into collision with a horse and springcart driven by Thomas Slater, who was accompanied by a boy named Frederick Johnson. With a sudden impact the cart swung round, and Slater was thrown head first over the front board of the trap, falling with his horse between the wheels of the wagon. One of the wheels passed over the heel of Slater's right boot, completely crushing it and seriously injuring the foot, and then passed over one of the fetlocks of his horse, severing the hoof from the leg. The scene was one of indescribable confusion. Slater was dragged from underneath the wagon, and it was then found that he had also sustained an injury to his left knee. The horse, limping, was conducted over the bridge and shot. Two railings of the bridge were broken, and it was only the wires that prevented the horse and cart from tumbling over. The boy Johnson managed to remain in the cart all the time, and he escaped without injury."* (Evening Mail, 1895)

In the NAASRA (National Association of Australian State Road Authorities) Highway Bridge Design Specification of 1965, there are design requirements for roadway railings on bridges, for footway railings on bridges, and for "crash resisting railings" on bridges. Even in 1965, barriers were only designed to actually resist impact loads from vehicles on, "bridge structures carrying traffic over busy thoroughfares", otherwise design loads were approximately 2 kN/m. In 1992, the AUSTROADS Bridge Design Code came into effect, and barrier loads jumped to 90kN. In 2004 a new Australian Standard for Bridge Design, AS 5100 introduced a design load up to 500kN to resist heavy vehicles. In the 2017 revision of AS 5100 this increased again to 600kN.

5.5.2 *Design of New Steel Barriers*

Timber ordnance railings are well recognised as having grossly deficient impact capacity when compared to the current design loadings. Furthermore they have hazardous features such as the impalement risk from dislodged timbers and do not meet current code requirements for pedestrians. For these reasons, when bridges are rehabilitated, new steel barriers are provided.

Timber truss bridges will not withstand impact loads specified in current design standards, and so barriers must be designed to a lower standard. Generally, barriers are designed for 90kN. Barriers have a considerable visual impact on the bridge, as well as increasing the dead load significantly. Care needs to be taken to ensure that these barriers will actually perform by providing well designed and robust load paths, so that both the bridges and the public are safe.

Crashes on timber bridges generally occur at a transition – either the transition from the road approach to the bridge (where there is a taper), or the transition between the wider approach spans to the narrower truss span (where, again, there is often a taper). By eliminating or moving the taper away from the truss span, the risk of an errant vehicle hitting the truss span is reduced.

5.5.2.1 Barrier Load Paths - Horizontal

Timber truss bridges do not have sufficient lateral strength or stiffness to withstand lateral loads from vehicle impacts. An upgraded barrier should not be connected to a truss which is unable to withstand the loads. This means that either the truss must be strengthened in order to take the lateral loads for which the barrier is designed, or an alternative load path must be provided. If a load path is decided upon (eg, barrier rails to barrier posts to steel cross girders to SLT deck to abutment), then the connections at each link need to be designed to reliably transfer the loads. If the load is to be taken from the steel cross girder into an SLT deck, then the normal sliding connections alone (provided to allow restressing of the deck, which is an absolute necessity for SLT decks) cannot be used for the SLT deck, either at the cross girders or at the abutment.

Similarly, the barriers must be detailed to avoid any unwanted load paths. If there is insufficient gap between the barrier post and the truss, then impact loads will go directly into the truss timbers, which will most likely result in them popping out under impact (top and bottom connections in timber diagonals have very limited lateral capacity), thereby destroying the truss.

5.5.2.2 Barrier Load Paths - Vertical

In addition to checking the load path for impact loads, the truss must be checked for the additional dead load due to the barrier. The type of steel barrier that is generally used in timber truss strengthening projects is significantly heavier than the original timber ordnance fencing.

5.5.2.3 Barrier Post Spacing

Barrier posts should be consistently spaced for the length of the bridge so as to minimise negative visual impacts. Posts must be clear of stressing strand locations where an SLT deck is present, and care should be taken to provide a visual relationship between the piers and the posts.

5.5.2.4 Other Considerations for Traffic Barriers

Traffic barriers need to be detailed so that they are maintainable (replacement of components will be necessary after traffic incidents) and they need to improve safety for road users including pedestrians and cyclists, while also conserving the heritage values of the bridge. In the past, attempts have been made to somewhat replicate the original barriers but in upsized steel. Unfortunately, this tends to increase the bulk of the rail many times over, and therefore obscures views of the truss. Having architectural input for the design of the traffic barriers for Monkerai Bridge demonstrated that using two much larger rails more closely spaced actually allows the two horizontal rails to read as a single element, thus simplifying the visual composition, reducing the complexity of the viewing plane, and making the heritage timber truss more visually dominant.

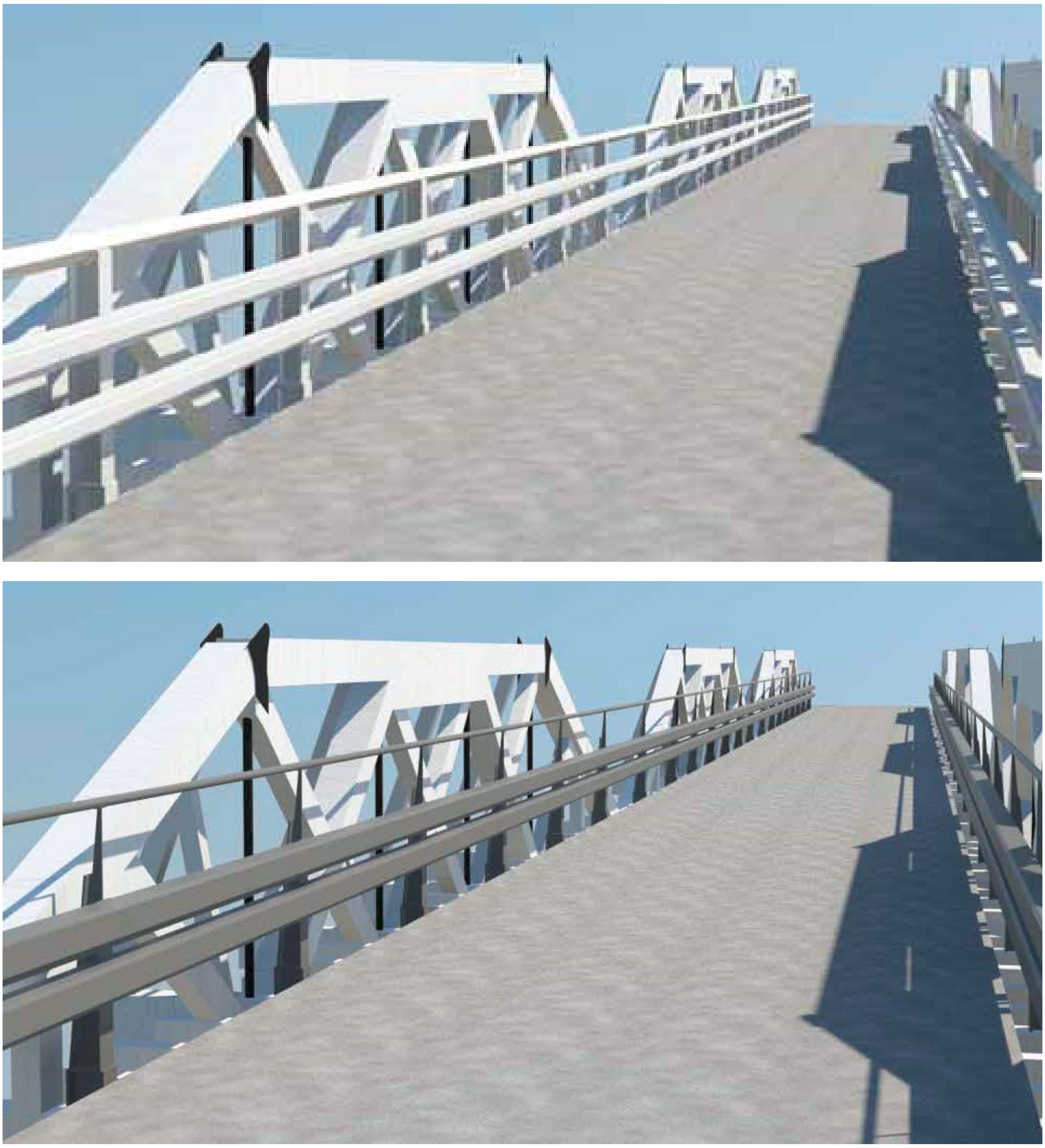


Figure 152: Top: "ordnance style" steel barrier, bottom: architect designed barrier for Old PWD truss

The original colour scheme for timber truss bridges was not simply a matter of aesthetics. White paint provides the best protection for timber against the damaging effects of ultraviolet radiation and it also absorbs less heat and so minimises the effects of accelerated drying of the timber. In addition to providing superior durability, the use of white paint on kerbs and railings was important to provide clear delineation for travellers crossing especially the narrow rural bridges.

For these reasons, timber truss bridges were originally always painted white, at least above the level of the deck. Sometimes the bottom chords, cross girders, and even the outer approach span timber girders and corbels were painted white. Metal elements including castings, tension rods, and sway bracing also require paint for the purposes of durability. For metal, however the colour is not important as the purpose of the paint is to protect the metal from oxygen and water rather than UV and heat. Metal elements in timber trusses were generally painted dark grey or black.

During the First and Second World War there was a policy of painting timber bridges grey so that they would be less conspicuous, but this only applied to bridges that were scheduled for repainting during those years. By 1933, general practice was to repaint timber on bridges silver-grey except the handrails and tops of kerbs were still painted white for traffic safety reasons.

As noted in Bridge Aesthetics (Bridge Aesthetics: Design guideline to improve the appearance of bridges in NSW, RMS 2012), "The use of white on old and modified timber bridges has become favoured as a distinguishing characteristic of such bridges and marks them well in the landscapes in which they sit." This same guideline also notes that, "A white painted finish can help emphasise the main features of a bridge." As noted in The Aesthetics of Bridges (The Aesthetics of Bridges: A Reference Manual for Bridge Designers, DMR 1987), pure white as a colour scheme needs to be treated with caution – "it can be very 'intrusive"'. This is true for timber painted white, but even more important for steel, which reflects still more light and almost glows.

It is clear that white is by far the most appropriate colour for timber in timber truss bridges, being the original colour and also the colour providing the best protection for the timber.

However painting new steel barriers on timber truss bridges white is less satisfactory. The colour of barriers where they may be touched by pedestrians does need to be a light shade (dark coloured metal becomes too hot to touch in the summer), however, white does create a tendency for the barriers to visually overpower the truss. Rather than white being used to "emphasise the main features of a bridge" which should be the truss, it often emphasises a new steel barrier.

This can perhaps be seen on the following page, where the same barrier system has been used on two bridges (Coonamit and Abercrombie), on one it is galvanised, and on the other it is painted white. Clearly, the barrier is obtrusive, ugly, and incredibly chunky, a poor visual outcome on both bridges, blocking views both to the truss and to the river. However, at Coonamit, where the barrier is a little darker than the truss, the truss is emphasised by its white colour. At Abercrombie, where the barrier is white, the truss is almost completely overwhelmed by the strong horizontal lines of the bright white barrier system, which is an unfortunately outcome.

It is therefore preferable to galvanise traffic barriers and have them painted RMS Bridge Grey.

Figure 153: Barriers different darker colour from truss, thereby letting truss draw the attention

Figure 154: Barriers of same style as above, but more attention drawn from truss due to colour

6 Worked Examples

6.1 A de Burgh Truss – Barham Bridge

6.1.1 Heritage Significance

Figure 155: Construction of Barham Bridge (University of Melbourne Archives Image Collection)

Barham Bridge crosses the Murray River with two de Burgh trusses flanking a lattice steel lift span founded on cast iron columns, and with a single timber girder approach span at each end of the bridge. The bridge is limited to a single lane over the lift span, but the timber truss spans and timber girder approach spans carry two lanes of traffic and a pedestrian walkway. As a gateway to NSW and the town of Barham, the bridge has social and aesthetic significance. The lift span continues to operate to this day, which has the effect of increasing community awareness of and interest in the bridge. The Statement of Significance, as listed on the RMS Section 170 Heritage and Conservation Register is very brief: "Barham Bridge, completed in 1904, is of State significance. The form and setting have aesthetic and social significance."

6.1.2 Design Actions

In accordance the provisions of Section 5.1, the following are used to determine design actions:

Dead Load:
Timber (not SLT): 11kN/m^3; Load Factor = 1.4
Timber (SLT): 11kN/m^3; Load Factor = 1.2
Metal: 78.5kN/m^3; Load Factor = 1.1
Live Load:
T44; DLA = 1.25; Load Factor = 2

A three dimensional model of a single truss span is prepared. Vertical struts are fixed at the base but pinned at the top. Top chords and bottom chords are continuous. Tension rods are pinned at both ends. Tension rods are defined as tension-only members and vertical struts are compression-only members. The end panels of the bottom chords may take compression or tension. One end of the span is fixed in position and the other is free to move longitudinally. Both ends are pinned. Critical axial and bending actions are given in Figs 156&157 respectively.

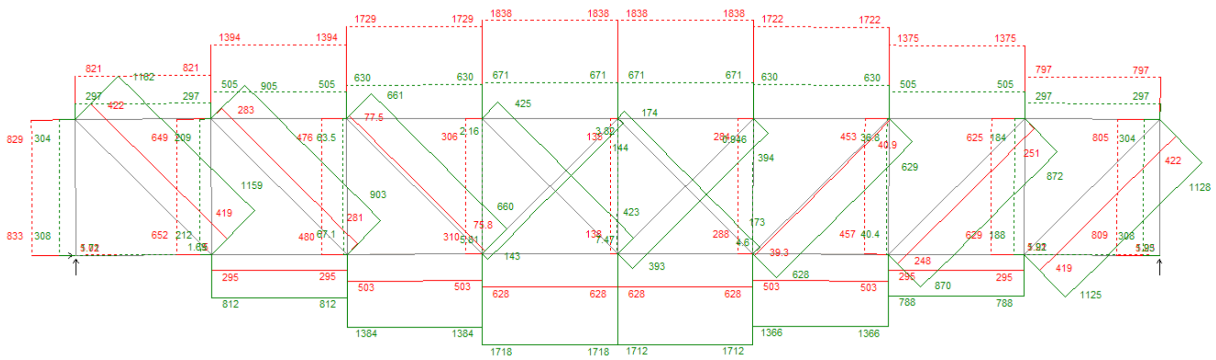


Figure 156: Design Actions (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

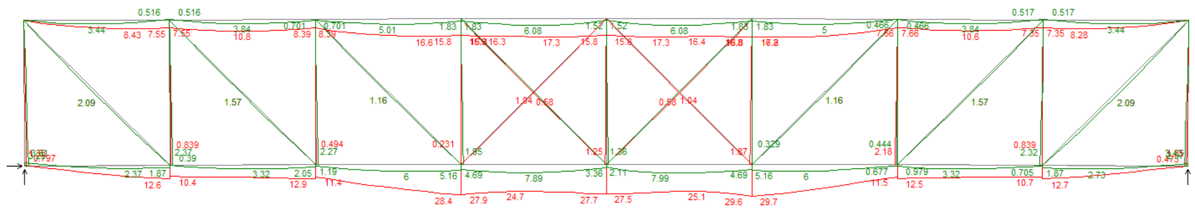


Figure 157: Design Action (Bending Moment Envelope): Ultimate Dead Load + Ultimate T44 Live Load

A summary of the maximum ultimate design actions for the timber member is given in Table 31.

Table 31: Approximate Maximum Ultimate Design Actions for Timber Members		
	Compression (kN)	Bending (kNm)
Top Chord	1850	20
Terminal Strut	850	5.0
1st Intermediate Strut	650	2.5
2nd Intermediate Strut	500	2.5
3rd Intermediate Strut	350	2.5
4th Intermediate Strut	150	2.5

6.1.3 Calculating Capacities of Vertical Struts

In order to calculate the capacity of a timber member subject to combined compression and bending, the simplification of the biaxial bending formula from Section 5.4.3.1 can be used:

$$\left(\frac{M^*_y}{M_{d,y}}\right)+\left(\frac{N^*_c}{N_{d,cy}}\right)\leq 1.0$$

The capacity in bending of a single flitch can now be calculated as follows:

$M_d \geq M^*$ where $M_d = \phi\, k_1\, k_{11}\, k_{21}\, f'_b\, Z$

M_d	=	design capacity in bending
M^*	=	design action effect in bending o for ultimate gravity loads, moment due to the axial forces in curved members must be calculated and added to forces due to fabrication. o Apply a factor of 0.5 to bending moments due to fabrication where fabrication occurs a minimum of 12 months prior to installation of timbers into the bridge as discussed in Section 5.4.1.1.
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor o 0.57 for permanent effects alone o 0.80 for serviceability live load (T44, Load Factor=1, DLA=25%) o 0.97 for ultimate live loads (T44, Load Factor=2.0, DLA=25%)
k_{11}	=	size factor = 0.997 for timber with maximum dimension 305 mm
k_{21}	=	shaving factor = 1.0 for sawn timber
f'_b	=	characteristic value in bending (55MPa for F22)
Z	=	section modulus which equals $db^2/6$ for bending about minor axis

The various bending moments to be considered are summarised in Table 32. Secondary bending moments are calculated by multiplying the axial load to be taken by a single flitch by the eccentricity of load due to curvature (in this case, say about 5 mm). Bending moments due to fabrication are calculated in accordance with the formula provided in Section 5.4.1.1, where the three values of E are $E_{0.05}$ (= 8,000 MPa), $E_{average}$ (= 16,000 MPa) and $E_{0.95}$ (= 24,000 MPa). Bending moments due to global behaviour are not included as they are minimum where other moments are maximum (ie, at the location of the central timber spacer) and are not critical.

Table 32: Bending Moments and Capacities of Vertical Struts

	Secondary	Fabrication (low E)	Fabrication (average E)	Fabrication (high E)	Strength
Terminal Strut	2	5.0	10	15	32
1st Intermediate	2	2.5	5.0	7.5	20
2nd Intermediate	1	2.5	5.0	7.5	20
3rd Intermediate	1	2.5	5.0	7.5	20
4th Intermediate	0	2.5	5.0	7.5	20

The capacity in compression of a single flitch can now be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\ k_1\ f'_c\ A_c$; or
$= \phi\ N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor o 0.57 for permanent effects alone o 0.80 for serviceability live load (T44, Load Factor=1, DLA=25%) o 0.97 for ultimate live loads (T44, Load Factor=2.0, DLA=25%)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a single flitch
N_{cr}	=	0.5 x critical elastic buckling load of the column assembly o The critical elastic buckling load can be found using the method outlined in Section 4.6.3.4, but with the three values of E. o The model needs to reflect any changes with the new design (eg, steel cross girder and use of M24 bolts rather than 22mm diameter bolts).

A critical elastic buckling analysis is undertaken to obtain the appropriate values for N_{cr}. All bolts are M24 bolts (metric) rather than the original M22 (imperial). Because bolts are in pairs, a bolt of 28 mm diameter is included in the model, which has equivalent stiffness ($I = \pi d^4/64$) to a pair of M24 bolts. The analysis for each member is conducted three times with three different values of E (8,000, 16,000, 24,000 MPa). Original locations of bolts and timber spacers are conserved.

Models are shown in Figure 158 on the following page. Results are shown in Table 33 below:

Table 33: Compressive Capacities of Vertical Struts (values given for single flitches or half assemblies)

	Compression Force / Flitch	Strength (low E)	Strength (average E)	Strength (high E)	Strength (material)
Terminal Strut	425	635	1140	1650	1165
1st Intermediate	325	425	730	1015	930
2nd Intermediate	250	425	730	1015	930
3rd Intermediate	175	425	730	1015	930
4th Intermediate	75	425	730	1015	930

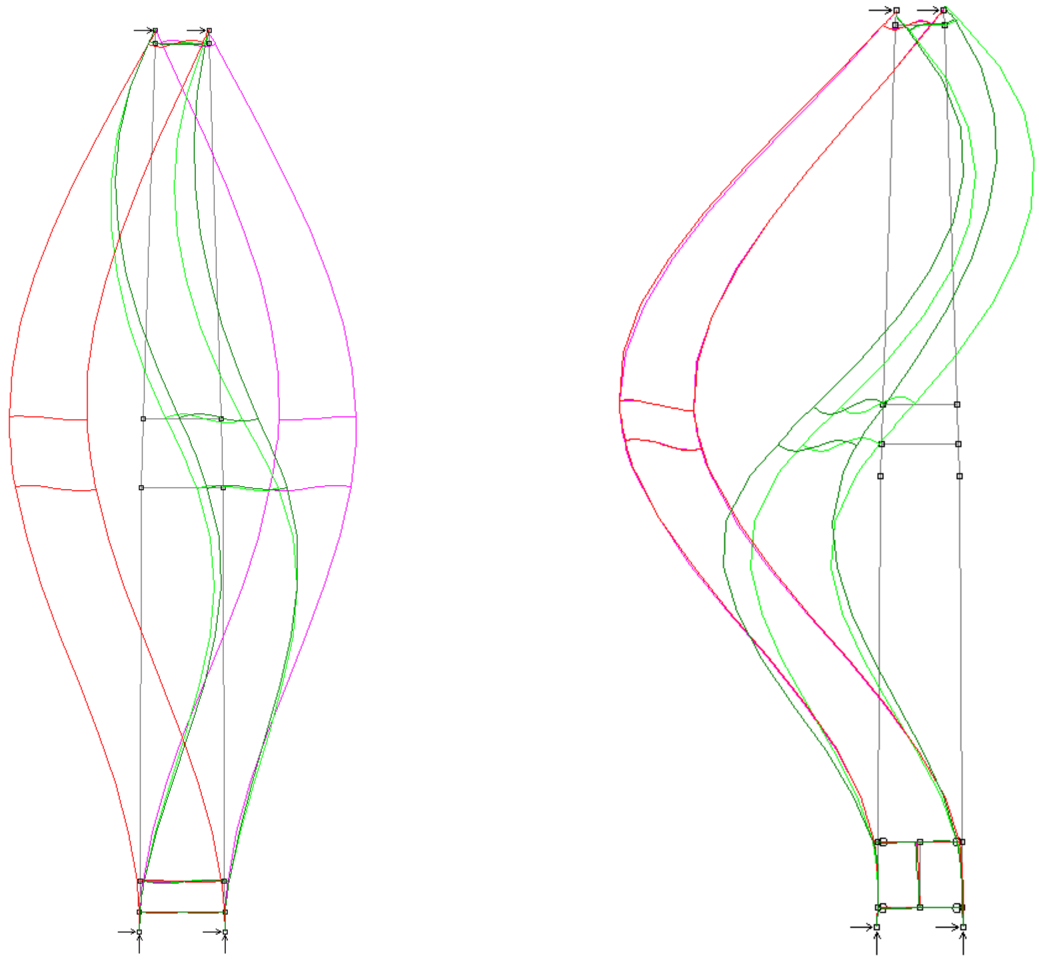


Figure 158: Model Terminal Strut (left); Model Intermediate Struts (right)

Now bending strengths and compressive strengths have been determined, they can be combined:

Table 34: Summary of Results for Combined Bending and Compression of Vertical Struts			
$\left(\frac{M^*_y}{M_{d,y}}\right)+\left(\frac{N^*_c}{N_{d,cy}}\right)\leq 1.0$			
	Low E (8,000 MPa)	Average E (16,000 MPa)	High E (24,000 MPa)
Terminal Strut	0.22 + 0.67 = 0.89	0.38 + 0.37 = 0.75	0.53 + 0.36 = 0.89
1st Intermediate	0.23 + 0.76 = 0.99	0.35 + 0.45 = 0.80	0.48 + 0.35 = 0.83
2nd Intermediate	0.18 + 0.59 = 0.77	0.30 + 0.34 = 0.64	0.43 + 0.27 = 0.70
3rd Intermediate	0.18 + 0.41 = 0.59	0.30 + 0.24 = 0.54	0.43 + 0.19 = 0.62
4th Intermediate	0.13 + 0.18 = 0.31	0.25 + 0.10 = 0.35	0.38 + 0.08 = 0.46

6.1.4 Recommendations for Vertical Struts

Therefore, changes from original details to meet current design loads are limited to the following:

- Increase bolt size from 22 mm (imperial) to M24 bolts (metric)

6.1.5 Calculating Capacity of the Top Chord

Similarly, the capacity of the top chord subject to compression can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\, k_1\, f'_c\, A_c$; or
$= \phi\, N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression (1850 kN)
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor o 0.57 for permanent effects alone o 0.80 for serviceability live load (T44, Load Factor=1, DLA=25%) o 0.97 for ultimate live loads (T44, Load Factor=2.0, DLA=25%)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a timber in top chord
N_{cr}	=	critical elastic buckling load of the assembly (lowest 5th percentile value of E (8,000 MPa) is critical so other values need not be checked)

An essential factor in determining the critical elastic buckling load of the assembly is the location and effectiveness of lateral restraints. The original sway bracing in the de Burgh trusses at Barham consisted of four 6” x 3” x 1/2” T sections per truss connected to the top chord with two 7/8” bolts in packed slotted holes. The bolts penetrate both top chord timber flitches, and pass through 2” (oversize) holes in the cast shoes. It has been common practice in timber truss bridges to increase the number of sway braces so that lateral restraint can be provided at each panel point. This is as important aspect of strengthening the top chord, but in order to be successful, not only the number, but also the effectiveness of these restraints must be increased.

The method for estimating the design force on each lateral restraint in found in Section 5.4.2.2:

$$N^*_R = \frac{0.1 N^*_c}{n+1} h_{26} h_{27} g_{38}$$

where:

N^*_c	=	design force (approx 1850kN)
n	=	number of intermediate restraints (=7)
h_{26}	=	1.5 (unseasoned with DL+LL)
h_{27}	=	1.0 (for sawn timbers)
g_{38}	=	1.0 (for restraining one top chord)
$\therefore N^*_R$	=	35kN

The angle of sway bracing is approximately 18°, and so the force in the sway bracing to provide a lateral restraint of 35kN is approximately 115kN, and this, in turn, requires a connection capacity at the top chord of 105kN, which is about 8 times the original capacity. Changing the sway bracing to a stronger section will not assist in providing additional lateral restraint unless there is also a change in the way that the forces are transferred from the top chord into the sway bracing.

The capacity of a metal to metal bolted connection is more than an order of magnitude greater than that of a metal to timber bolted connection. For the sway bracing to be made effective, it must be connected directly to a metal element rather than to timber. This can be achieved with relative simplicity in a de Burgh truss due to the fact that there is already a stiffening plate in the top chord shoe at a convenient location. With minor modifications, a bolted connection can be achieved directly, as demonstrated diagrammatically in Figure 159 below.

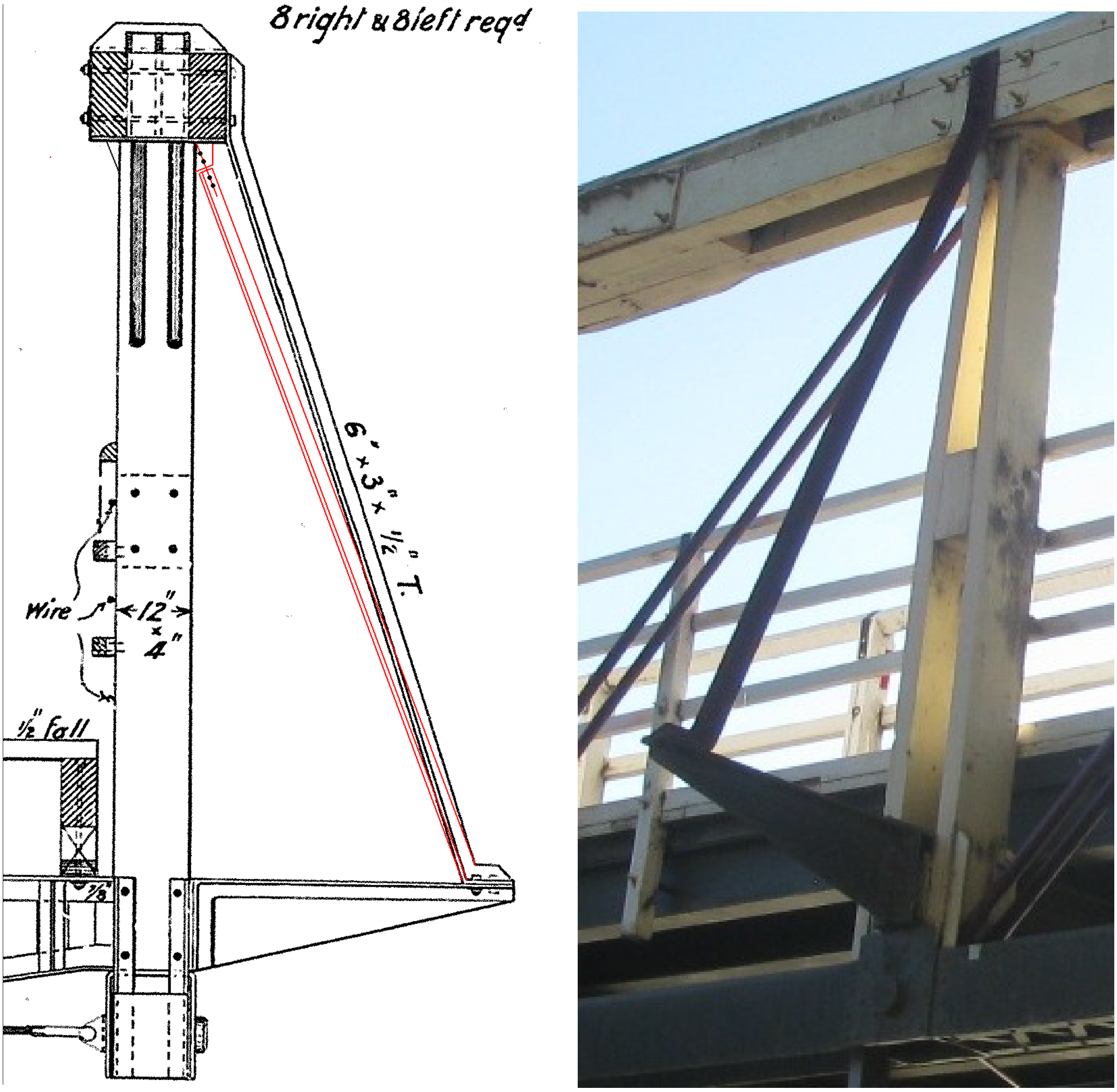


Figure 159: Modifications to Top Chord Shoes and Sway Braces to Achieve Lateral Restraint

A similar process is used to determine the critical buckling load as previously described. A model of the top chord including the two flitches, intermediate bolts and cast iron shoes is created as shown in Figure 160. Lateral restraints are provided at each panel point, and the vertical struts are also modelled to provide the vertical restraints at each panel point. The loads are applied as shown in Figure 161 to represent the variable load along the length of the top chord.

Figure 162 shows the buckling mode resulting from the ultimate dead and live load effects.

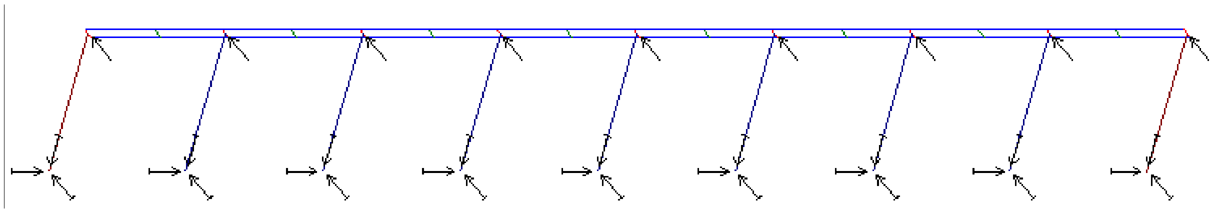


Figure 160: Microstran Model to Determine Elastic Critical Buckling Load of Top Chord

Figure 161: Application of Variable Load to Microstran Model

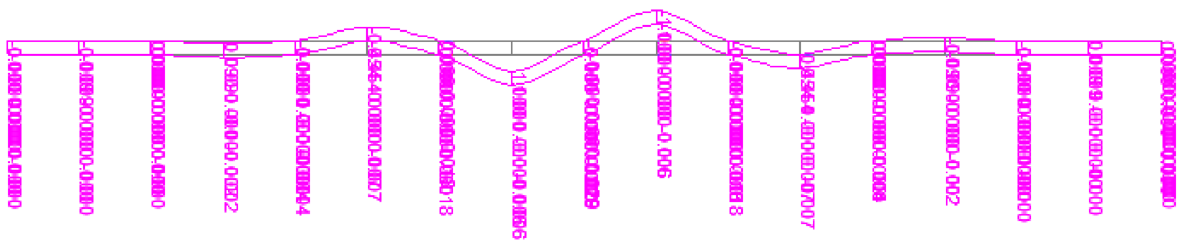


Figure 162: Buckling Mode Determined by Microstran Elastic Critical Buckling Analysis

From this Microstran Analysis, the value of N_{cr} for the top chord is determined to be 5075 kN. Therefore, the capacity of the top chord in compression is 3815 kN which is more than adequate.

As a vehicle travels across the bridge, the top chord has a tendency to bow inwards at the location of the vehicle due to bending of the cross girders. The extent of this bow depends on the stiffness of the cross girders. This bending of the top chord causes an eccentricity of the load path with associated bending moments. For the purposes of completeness, these bending moments should be considered in combination with the compressive stress in the top chord.

The lateral deflections due to bending of the cross girders is generally very small. For the ultimate limit state of combined dead load and live load, the maximum lateral deflection is 35 mm at centre span. The deflection is generally spread over a number of cross girders (Figure 162).

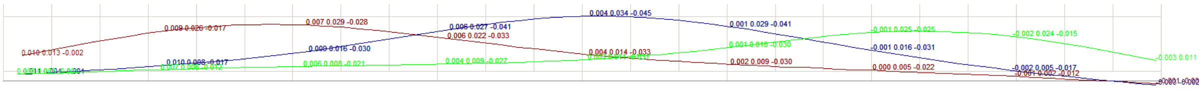


Figure 163: Inward Deflection of Top Chord for 3 Combined Load Cases Showing Distribution

If a design eccentricity of 50 mm is considered, then the maximum eccentricity for a single panel is approximately 20 mm due to the fact that the eccentricity is shared over a number of panels.

The design bending moment due to eccentricity is therefore 0.02 x 1838 = 37 kNm

The design capacity for bending about the minor axis is 2 x 0.75 x 0.97 x 55 x 1.8736 = 150 kNm

The design capacity for bending about the major axis is 2 x 0.75 x 0.97^2 x 55 x 3.7472 = 290 kNm

We can therefore verify that the top chord has sufficient capacity under biaxial bending and compression using the following conservative criteria provided in AS1720.1 Appendix E:

$$\left(\frac{M^*_y}{M_{d,y}}\right)^2 + \left(\frac{M^*_x}{M_{d,x}}\right) + \left(\frac{N^*_c}{N_{d,cy}}\right) \leq 1.0$$

$$\left(\frac{M^*_y}{M_{d,y}}\right) + \left(\frac{M^*_x}{M_{d,x}}\right)^2 + \left(\frac{N^*_c}{N_{d,cy}}\right) \leq 1.0$$

Where:

M^*_y = 37 kNm
$M_{d,y}$ = 150 kNm
M^*_x = 20 kNm
$M_{d,x}$ = 290 kNm
N^*_c = 1838 kN
$N_{d,c}$ = 3815 kN

$0.25^2 + 0.07 + 0.48 = 0.61 < 1$ – therefore OK
$0.25 + 0.07^2 + 0.48 = 0.73 < 1$ – therefore OK

6.1.6 Recommendations for Top Chord

Therefore, changes from original details to meet current design loads are limited to the following:

- Enlarge stiffener on outer side of top chord castings to enable connection of sway bracing (this will generally require casting of new top chord castings with ductile cast iron, which is a sensible precaution anyway due to the fact that the original iron is subject to brittle fracture).
- Increase number and section size of sway bracing (sway braces required at all panel points).
- Change angle of sway bracing in order to align with shoe to form a new connection point.

6.2 A Dare Truss – Briner Bridge

6.2.1 Heritage Significance

Figure 164: Briner Bridge over the Upper Coldstream River

Briner Bridge over the Upper Coldstream River is located approximately 35 km northeast of Grafton. It consists of a single Dare type truss span, with a number of timber girder approach spans. It was named the Briner Bridge by Mrs Briner after her husband Mr. G. S. Briner M.L.A. at an opening ceremony which they both attended on Wednesday 5 August 1908. It is a single lane bridge along its entire length, and the horizontal alignment is such that a vehicle at one end of the bridge cannot see whether there is a vehicle coming the other way. Significant modifications to the bridge substructure have been undertaken, including the provision of large concrete platforms extending approximately 300mm above the water level at the four river piers.

The Statement of Significance, as listed on the RMS Section 170 Heritage and Conservation Register, includes the following information: "As a timber truss road bridge, it has strong associations with the expansion of the road network and economic activity throughout NSW, and Harvey Dare, the designer of this type of truss… The Briner bridge is a representative example of Dare timber truss road bridges, and is assessed as being Regionally significant, primarily on the basis of its technical and historical significance."

6.2.2 Design Actions

In accordance the provisions of Section 5.1, the following are used to determine design actions:

Dead Load:
Timber (not SLT): 11kN/m^3; Load Factor = 1.4
Timber (SLT): 11kN/m^3; Load Factor = 1.2
Metal: 78.5kN/m^3; Load Factor = 1.1
Live Load:
T44; DLA = 1.25; Load Factor = 2

A three dimensional model of a single truss span is prepared. Timber principals and diagonals are pinned at both ends. Top chords and bottom chords are continuous. Tension rods are pinned at both ends. Tension rods are defined as tension-only members and timber diagonals are compression-only members. One end of the span is fixed in position and the other is free to move longitudinally. All supports are pinned. Axial and bending envelopes are given below.

Since a stress laminated timber (SLT) deck is now a standard treatment for timber trusses, the model includes an SLT deck rather than the current timber stringers, transverse decking and longitudinal sheeting. Also, since barrier rails are to be upgraded to standard steel "ordinance style" barriers, the dead weight from such barriers is included along with steel cross girders which generally replace under-capacity timber cross girders and also enable rigid connections. The distance between kerbs is, by necessity, reduced with the introduction of an upgraded barrier rail, and so the T44 design vehicle is offset 600 mm from the new kerb to determine design actions. The new steel cross girders are assumed to consist of 2/380PFC welded toe to toe.

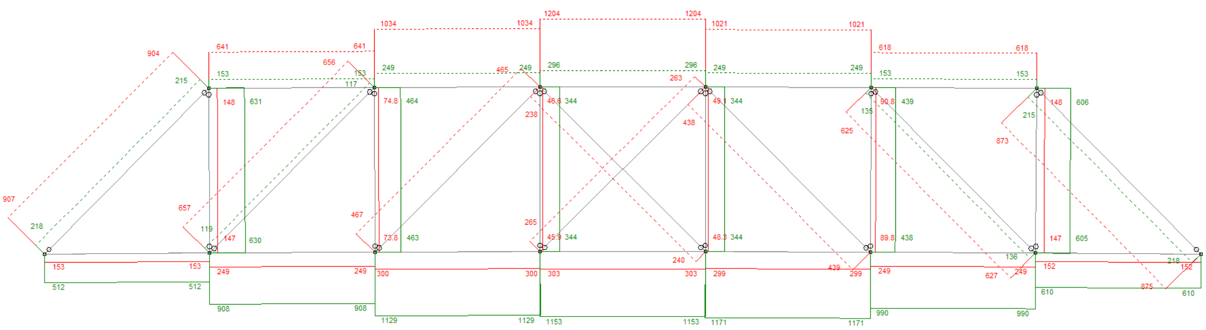


Figure 165: Design Actions (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

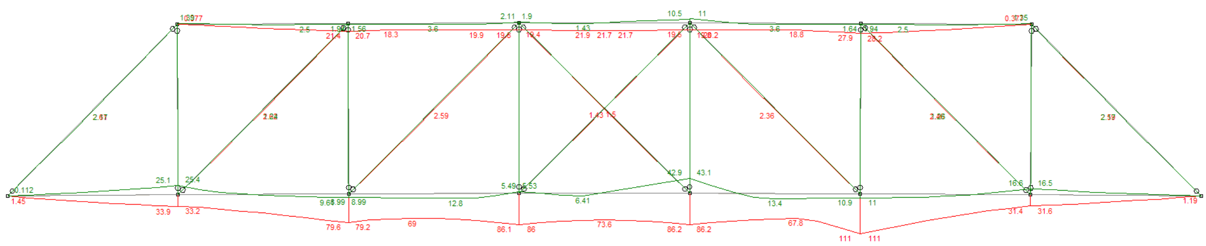


Figure 166: Design Action (Bending Moment Envelope): Ultimate Dead Load + Ultimate T44 Live Load

In order to accurately model the stiffness of a member in a timber truss, both the modulus of elasticity (E) and the second moment of area (I) must be given. For truss timber, the relevant modulus of elasticity is 16,000MPa (F22 timber). However, the second moment of area for an assembly of two members is more complex. If zero composite action is assumed, the model will underestimate the stiffness which may cause member buckling and instability in the model. However, if full composite action is assumed then the model will overestimate the stiffness, and this may attract unrealistic bending moments or otherwise alter the distribution of forces.

To determine a reasonable I value, an elastic critical buckling analysis can be undertaken. This gives an elastic critical buckling load N which can then be used to determine an equivalent value of I. Because the timber spacers provide inconsequential shear resistance, they should not be included in the Microstran model. However, the model should accurately reflect the distance between the two flitches, and the full number of bolts which connect the two flitches together. Supports should be pinned at each end, and load applied directly to the flitches.

The elastic critical buckling analysis shows the critical buckling mode, but can also calculate other possible buckling modes. For diagonal members in Dare trusses, the critical buckling mode is buckling of the whole member (shown in pink in Fig 167), which occurs at a significantly lower load that buckling of the flitches between the spacers (green in Fig 167). The lowest value of N is the relevant value to be used in determining an equivalent I value for a global model.

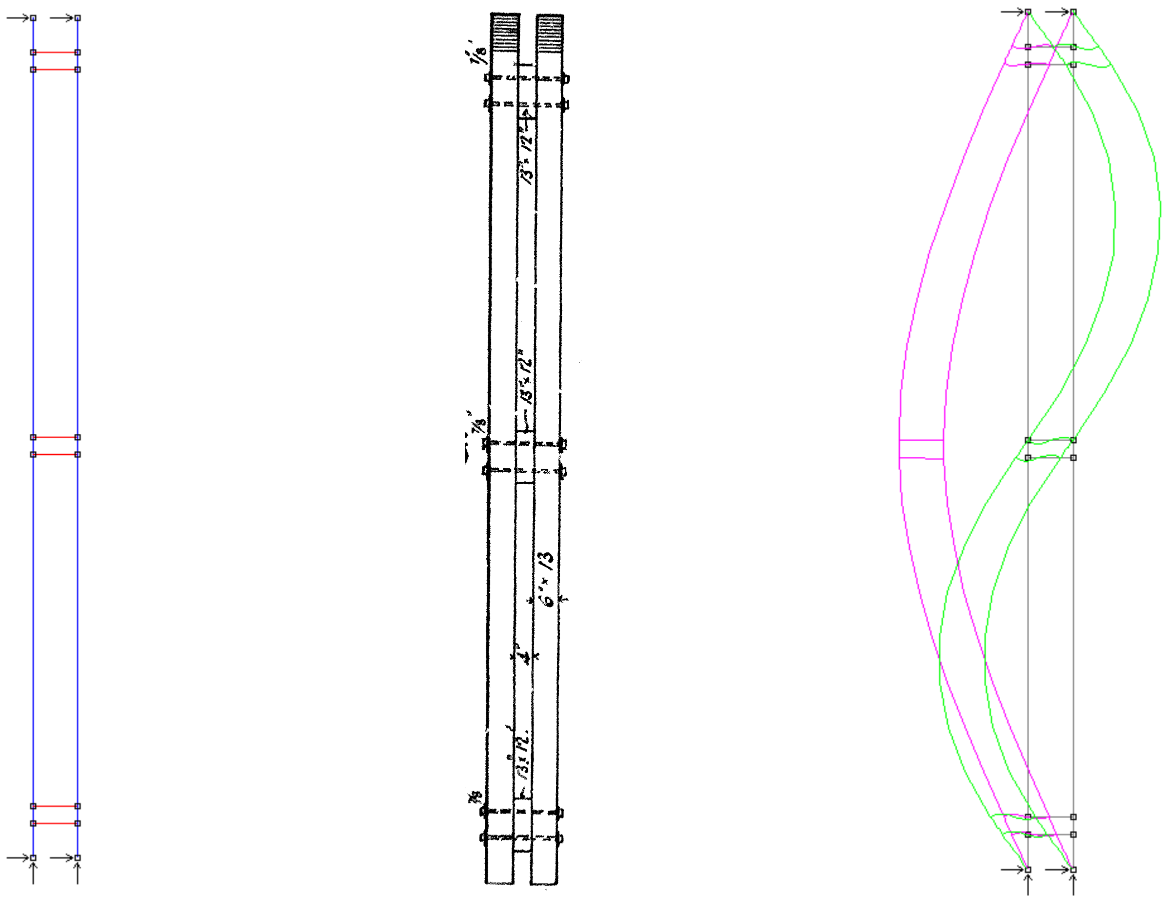


Figure 167: Model and Elastic Critical Buckling Analysis of 91' Dare Truss Principal

6.2.3 Calculating Capacities of Principals and Diagonals

The capacity in compression of a single flitch can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\, k_1\, f'_c\, A_c$; or
$= \phi\, N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor o 0.57 for permanent effects alone o 0.80 for serviceability live load (T44, Load Factor=1, DLA=25%) o 0.97 for ultimate live loads (T44, Load Factor=2.0, DLA=25%)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a single flitch
N_{cr}	=	0.5 x critical elastic buckling load of the column assembly o The critical elastic buckling load can be found as outlined on the previous page, but using the lowest 5th percentile value of E (= 8,000) and fixing the supports at the base to reduce the effective length. o The model needs to reflect any changes with the new design (eg, steel cross girder and use of M24 bolts rather than 22mm diameter bolts).

Table 35: Compressive Capacities of Assemblies (assuming all bolts through timber spacers to be M24)

	Design Force	Strength (buckling)	Strength (material)	
Brace G	910	1095	3020	OK
Brace H	660	470	1620	Fail
Brace J	470	350	1400	Fail
Brace K	265	350	1400	OK

6.2.4 Recommendations for Principals and Diagonals

Therefore, the first two diagonals require some form of strengthening. This is to be expected, as there is evidence from history that these two diagonals in Dare trusses tend to be overstressed.

In order to achieve sufficient capacity, the thickness of the timber flitches in both the first and second diagonal would have to be increased by approximately an inch (150 x 230 for the first diagonal and 140 x 200 for the second diagonal). This can be achieved with minimum modifications to the existing cast iron shoes by reducing the gap between the flitches. However, in order to allow for maintenance (particularly painting) and inspection, the gap between flitches should not be reduced to less than 50 mm (or preferably 75 mm). This may require casting of new top and bottom chord shoes with ductile cast iron, which is a sensible precaution anyway due to the fact that the original iron is subject to brittle fracture.

6.2.5 Calculating Capacity of the Top Chord

Similarly, the capacity of the top chord subject to compression can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\, k_1\, f'_c\, A_c$; or
$= \phi\, N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression (1200 kN)
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor o 0.57 for permanent effects alone o 0.80 for serviceability live load (T44, Load Factor=1, DLA=25%) o 0.97 for ultimate live loads (T44, Load Factor=2.0, DLA=25%)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a timber in top chord
N_{cr}	=	critical elastic buckling load of the assembly (lowest 5th percentile value of E (8,000 MPa) is critical so other values need not be checked)

An essential factor in determining the critical elastic buckling load of the assembly is the location and effectiveness of lateral restraints. The original sway bracing on the Dare truss at Briner Bridge consisted of four 6" x 3" x 1/2" T sections per truss connected to the top chord with two 7/8" bolts in packed slotted holes. The bolts penetrate both top chord timber flitches only. It has been common practice in timber truss bridges to increase the number of sway braces so that lateral restraint can be provided at each panel point. This is as important aspect of strengthening the top chord, but in order to be effective, not only the number, but also the effectiveness of these restraints must be increased. The critical aspect of effectiveness is the connections.

The method for estimating the design force on each lateral restraint in found in Section 5.4.2.2:

$$N^*_R = \frac{0.1 N^*_c}{n+1} h_{26} h_{27} g_{38}$$

where:

N^*_c	=	design force (approx 1200kN)
n	=	number of intermediate restraints (=4)
h_{26}	=	1.5 (unseasoned with DL+LL)
h_{27}	=	1.0 (for sawn timbers)
g_{38}	=	1.0 (for restraining one top chord)
$\therefore N^*_R$	=	36kN

The angle of sway bracing is approximately 20°, and so the force in the sway bracing to provide a lateral restraint of 36kN is approximately 115kN, and this, in turn, requires a connection capacity at the top chord of 100kN, which is about 8 times the original capacity. Changing the sway bracing to a stronger section will not assist in providing additional lateral restraint unless there is also a change in the way that the forces are transferred from the top chord into the sway bracing.

With the current sway bracing, which is ineffective in providing lateral restraint, one of the buckling modes in Fig 168 or 169 is expected. Both of these buckling modes are commonly seen in the top chords of Dare trusses, indicating that they are indeed under-capacity for current loads.

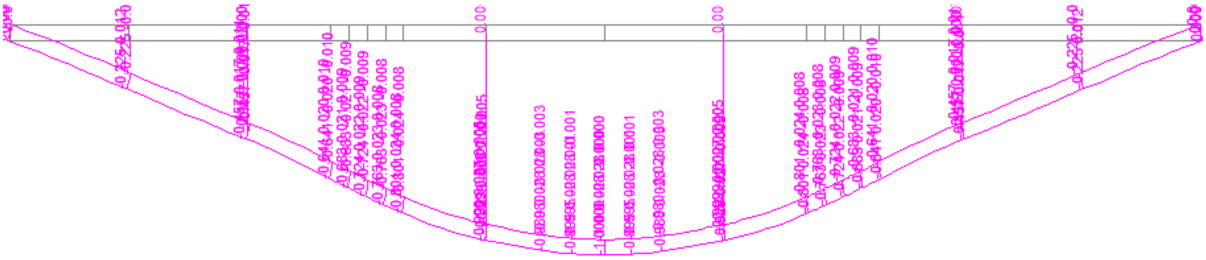


Figure 168: First Buckling Mode if sway braces are ineffective – Capacity 660kN

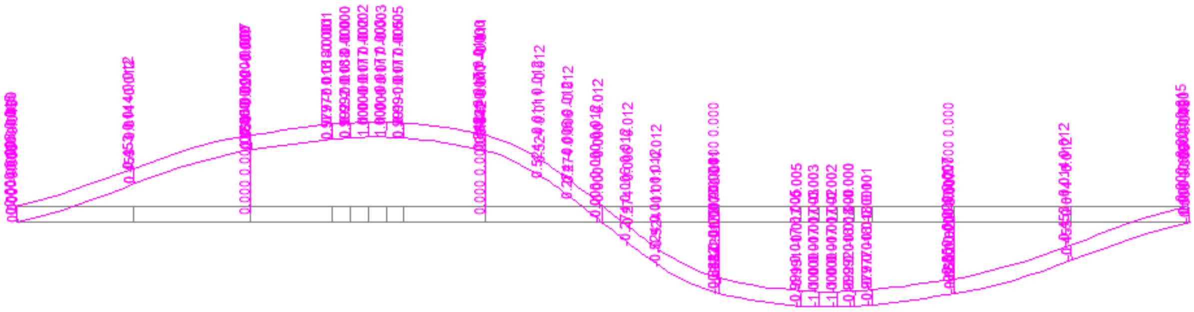


Figure 169: Second Buckling Mode if sway braces are ineffective – Capacity 680kN

If the sway bracing is made effective for provision of lateral restraint at every panel point, then the theoretical capacity increases to approximately 1900kN, which is more than sufficient.

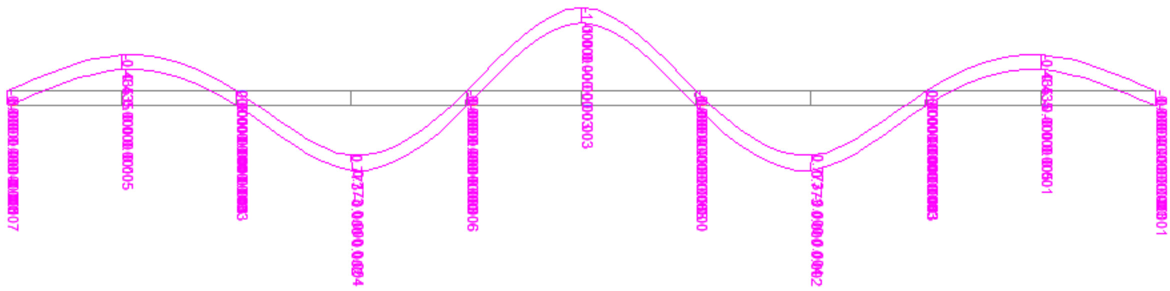


Figure 170: Buckling Mode if sway braces are made effective – Capacity 1900kN

6.2.6 *Recommendations for Top Chord*

Buckling of top chords in all five timber truss types is a known and common phenomenon, and it is therefore critical to guard against this risk. The deficiency in the sway bracing is not the sway bracing itself, but the method of connection to the top chord. In order to achieve sufficient capacity at the connection, the number of bolts per connection would increase from 2 to 16. There is clearly insufficient room at the interface between the sway bracing and the top chord to physically fit 16 bolts. If a plate were to be welded to the top of the sway bracing so that more bolts could be added, the plate would have to be almost 2m in length in order to achieve the necessary bolt spacing such that each bolt could carry sufficient load. Such large steel plates and such a large number of bolts would have a significant impact both visually and to durability.

One option which was used in the past to provide lateral restraint to timber truss bridge top chords was the addition of steel overhead portal bracing. The effectiveness of this bracing is dependent upon the stiffness of the steel portal, which means that they are of necessity quite large and do have a significant visual impact on the bridge from all common viewing angles.

Figure 171: Portal Bracing used in the past to provide lateral restraint to bowing top chords.

Although de Burgh trusses have conveniently shaped top chord castings such that a direct connection between the sway bracing and the casting is possible (thereby increasing capacity), unfortunately the detailing of shoes in the four remaining truss types precludes this option.

The simplest and most effective way of making the sway bracing effective for lateral restraint would be to add a knee brace to the sway bracing so that it no longer acts as a tension and compression member but begins to act directly to restrain lateral movement of the top chord.

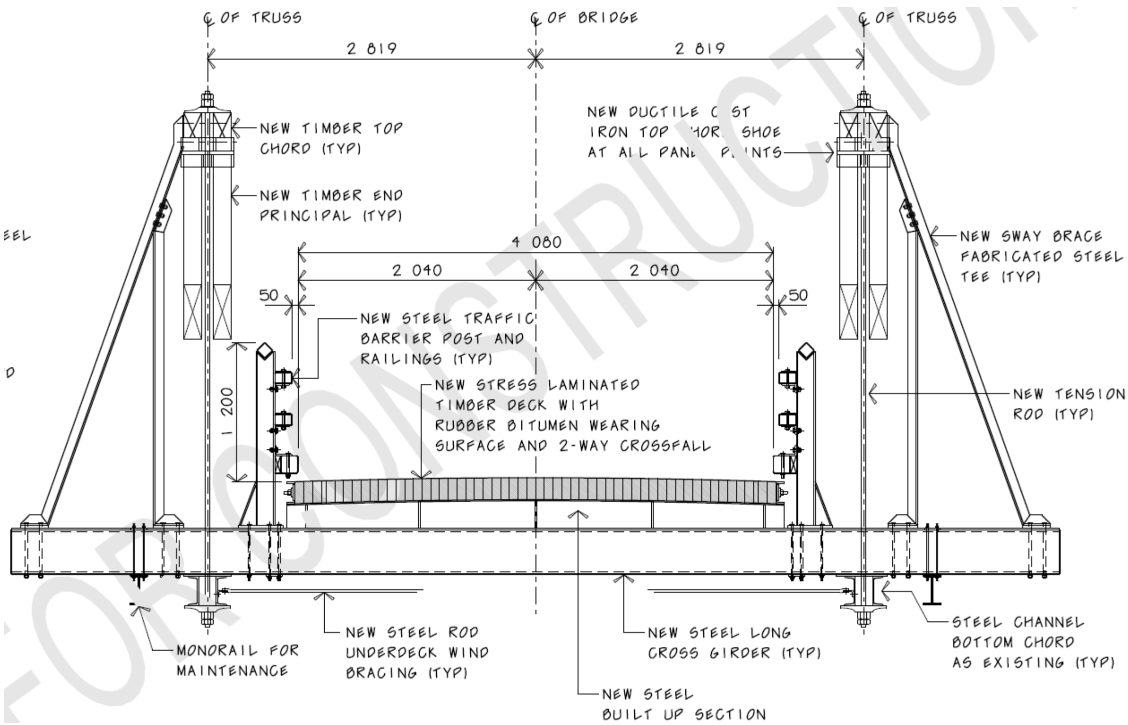


Figure 172: Making Sway with Vertical Knee Brace suitable for Allan, Dare and McDonald trusses

The top of the sway bracing (above the knee brace) now acts primarily in bending (and must be designed to have sufficient bending capacity to resist the lateral loads applied by the top chord). The part of the sway brace below the knee brace, and the knee brace itself, act in tension and compression (depending upon whether the top chord is trying to buckle inward or outward), and must be designed to have sufficient axial capacity to resist those tension and compression forces.

It is preferable, to minimise the visual impact, when strengthening the sway bracing, to increase the thickness of the steel rather than increasing the overall dimensions of the sway braces. This approach would also be suitable for strengthening sway braces in Allan and McDonald trusses. For an Old PWD truss, however, the original sway braces were timber, and so square hollow sections should be provided for both the sway braces and the knee braces in any upgrade.

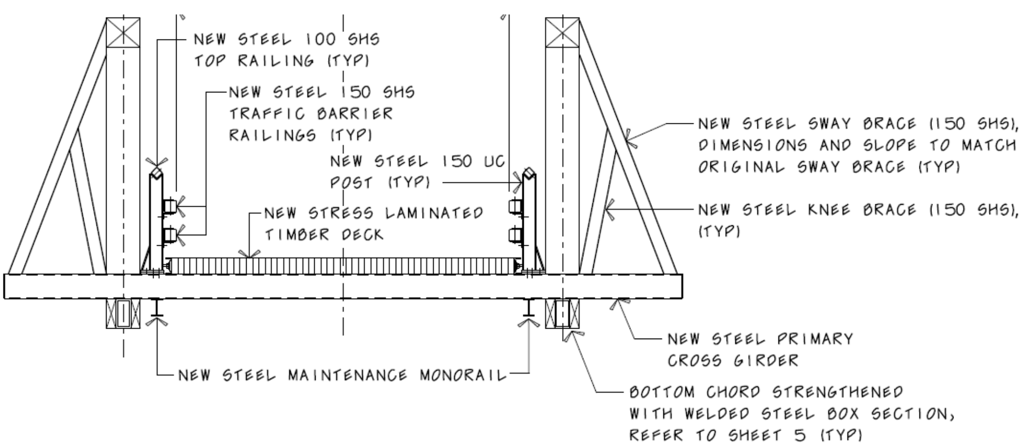


Figure 173: Making Sway Brace Effective by Addition of Vertical Knee Brace in Old PWD trusses

6.3 An Old PWD Truss – Clarence Town Bridge

6.3.1 Heritage Significance

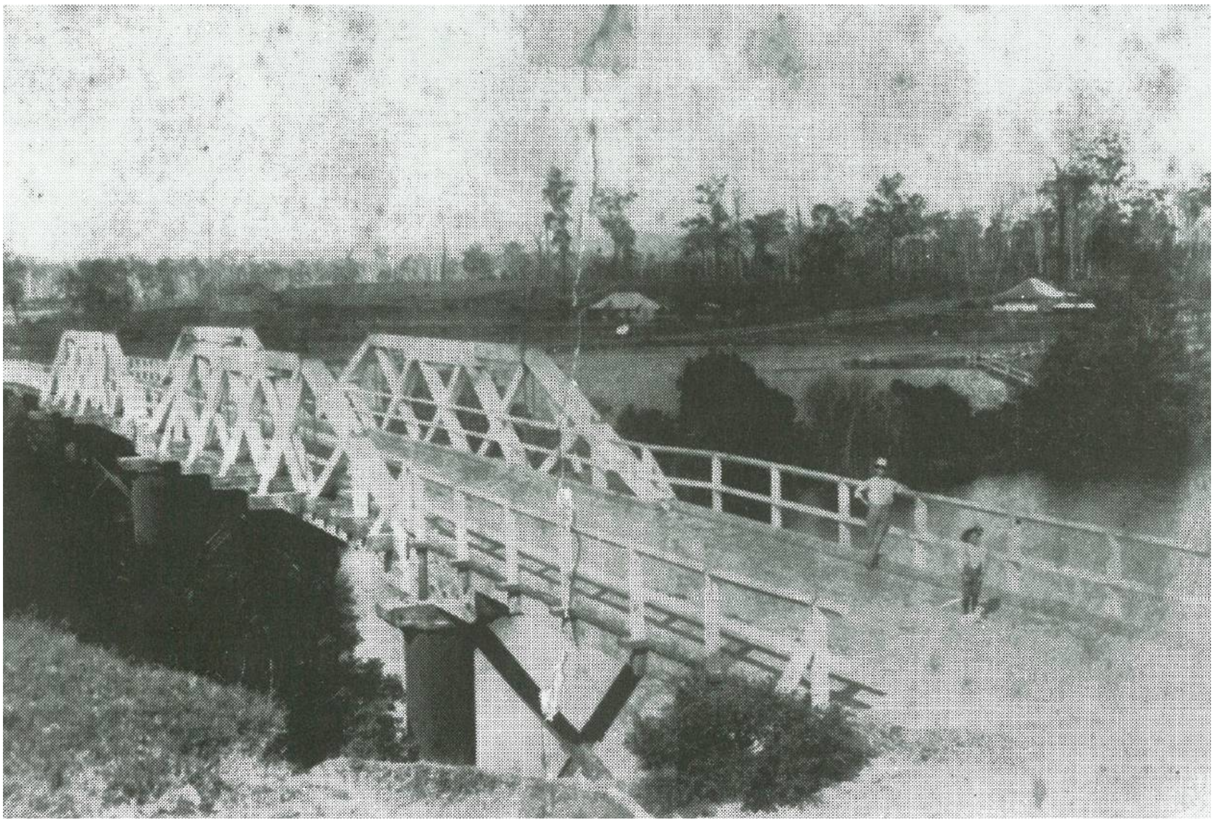


Figure 174: Clarence Town Bridge over the Williams River, as constructed in 1880

Clarence Town Bridge, officially named the "Brig O' Johnston" is the oldest remaining timber truss bridge in NSW, and crosses the Williams River with two 100' Old PWD trusses founded on cast iron columns, and with a single approach span at one end, and three approach spans at the other end. The bridge is limited to a single lane over the truss spans, but the approach spans carry two narrow lanes of traffic. The bridge has undergone a number of changes throughout its life, and many of these changes have had a negative impact on its load carrying capacity.

The first major changes occurred when the bridge was completely rebuilt in 1926-27. Although it was reported that the bridge trusses were, "completely rebuilt to their old design" it is clear that those responsible for rebuilding the bridge did not have a clear understanding of the original design. Two changes were introduced at this reconstruction (compare Figure 174 with Figure 175), both of which substantially reduced the capacity of the bridge. The first change was the introduction of metal sway braces, such as were provided in the four later truss types. These metal sway braces have only a small fraction of the capacity of the original timber sway braces to prevent lateral movement of the top chord. The second change was the introduction of a spliced laminated top chord rather than a single solid top chord. This change was probably made due to the difficulties in obtaining large long sections of timber, but it dramatically decreases capacity.

Figure 175: Clarence Town Bridge over the Williams River after reconstruction in 1926-27

It is very likely that the joint layout in the laminated timber bottom chord was also modified at this time. The original joint layout would have required a piece of timber exceeding 18 m in length, and a number of pieces over 12 m and 13 m in length. While this was feasible in the 1860s to early 1880s, it would have been highly improbable that such lengths could be obtained in the 1920s, especially since they seemed to be unable even to obtain suitable members for the top chords, which were only approximately 13 m in length. Using shorter lengths of timber, and increasing the number of joints in the laminated timber bottom chord again has a substantial negative impact on the strength, stiffness, and serviceability of an Old PWD truss.

Another change is the introduction of timber end posts at the abutments, which is again a very typical detail in later timber bridges, but was not originally included in Old PWD trusses. This change, however, is purely aesthetic, and does not impact the load carrying capacity of the bridge. The detailing of the attachment of the timber barrier posts to the approach span superstructure was also modified from the original detail to the detail commonly used in the later timber bridges.

One detail which was retained in the 1926-27 reconstruction was the very long timber girder approach spans, with the sawn outer girders aligning with the butting blocks of the truss spans, thereby providing some stability to the truss by taking some of the thrust from the principals back to the abutments, and in this manner relieving the under-strength timber bottom chord.

Unfortunately, the timber approach spans of 45' length were unable to withstand the increases in vehicle weights, and they were supplemented with steel girders in 1966. In 1974, the timber girders were removed, and with them, the load path back to the abutments also disappeared.

Clarence Town Bridge appears to be particularly susceptible to termite attack, and only 15 years after the bridge had been completely rebuilt, significant member replacements were required. Partial collapse of the bridge occurred in 1943 due to inadequate propping when an end principal was being replaced. As a result of the failure of the prop, laminates in the bottom chord fractured, and the truss also deflected downwards 475 mm. Complete collapse was only prevented by the rigidity of the deck system and the soundness of the upstream truss.

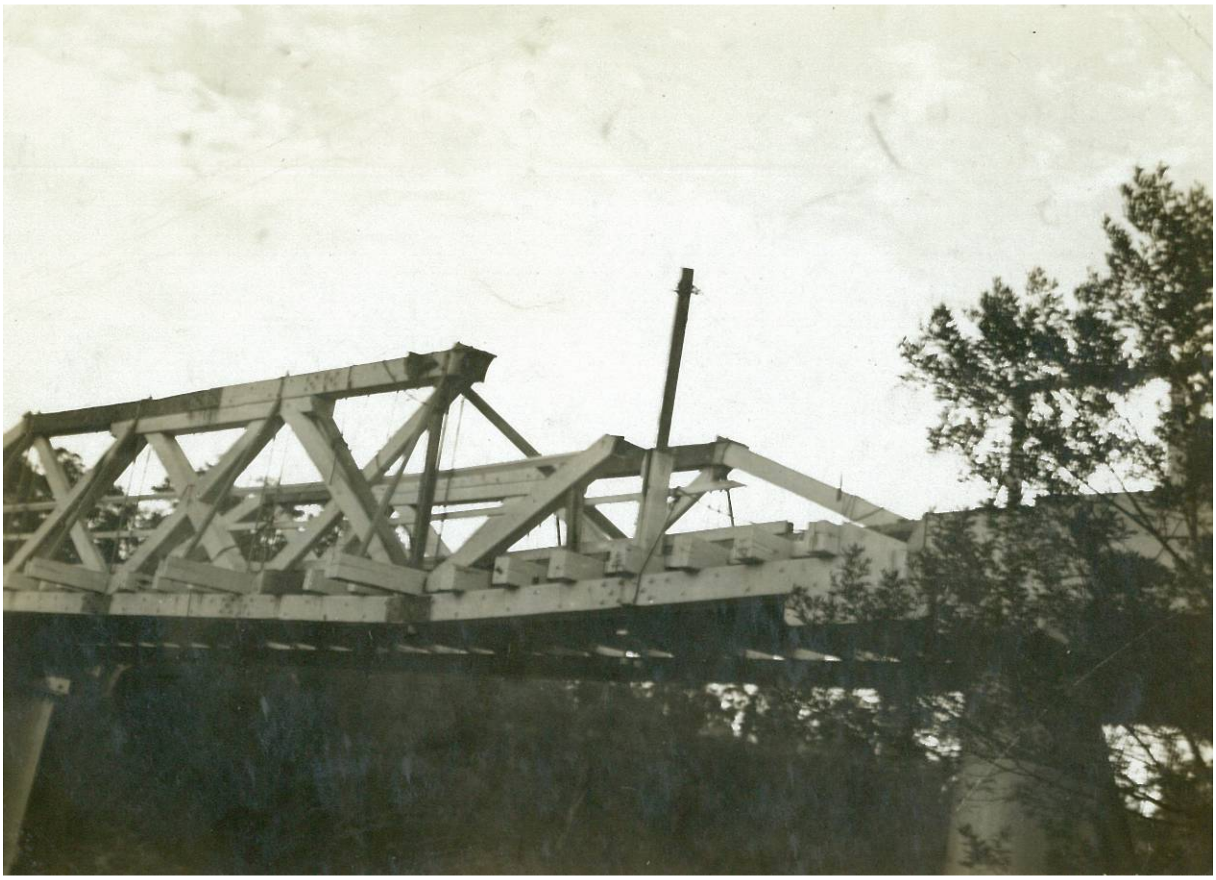


Figure 176: Clarence Town Bridge Bottom Chord Failure during Maintenance Works in 1943

From this experience, a new form of temporary support for member replacements in Old PWD and McDonald Trusses was invented in 1943, and first successfully trialled at Clarence Town. This support system (Figure 177) involved using gin poles, suspension cables and turnbuckles, and was later incorporated into DMR documentation such as the 1962 Bridge Maintenance Manual. It wasn't until the late 1940s that Bailey bridging became available and was successfully used to provide temporary support to timber truss bridges while members were being replaced. In the past decade or so, the legislative requirements for safety in the workplace have changed to such an extent that many of the traditional forms of timber bridge maintenance are no longer legal. The codes of practice for working at heights no longer permit activities such as climbing the ladders shown in Figure 177, or standing on the slats slung under the bridge while replacing elements. In addition to the risks of working at heights, the dangers of working in the vicinity of traffic are becoming more as traffic volumes continue to increase, and community expectations are such that it is generally not possible to close bridges for extended periods of time.

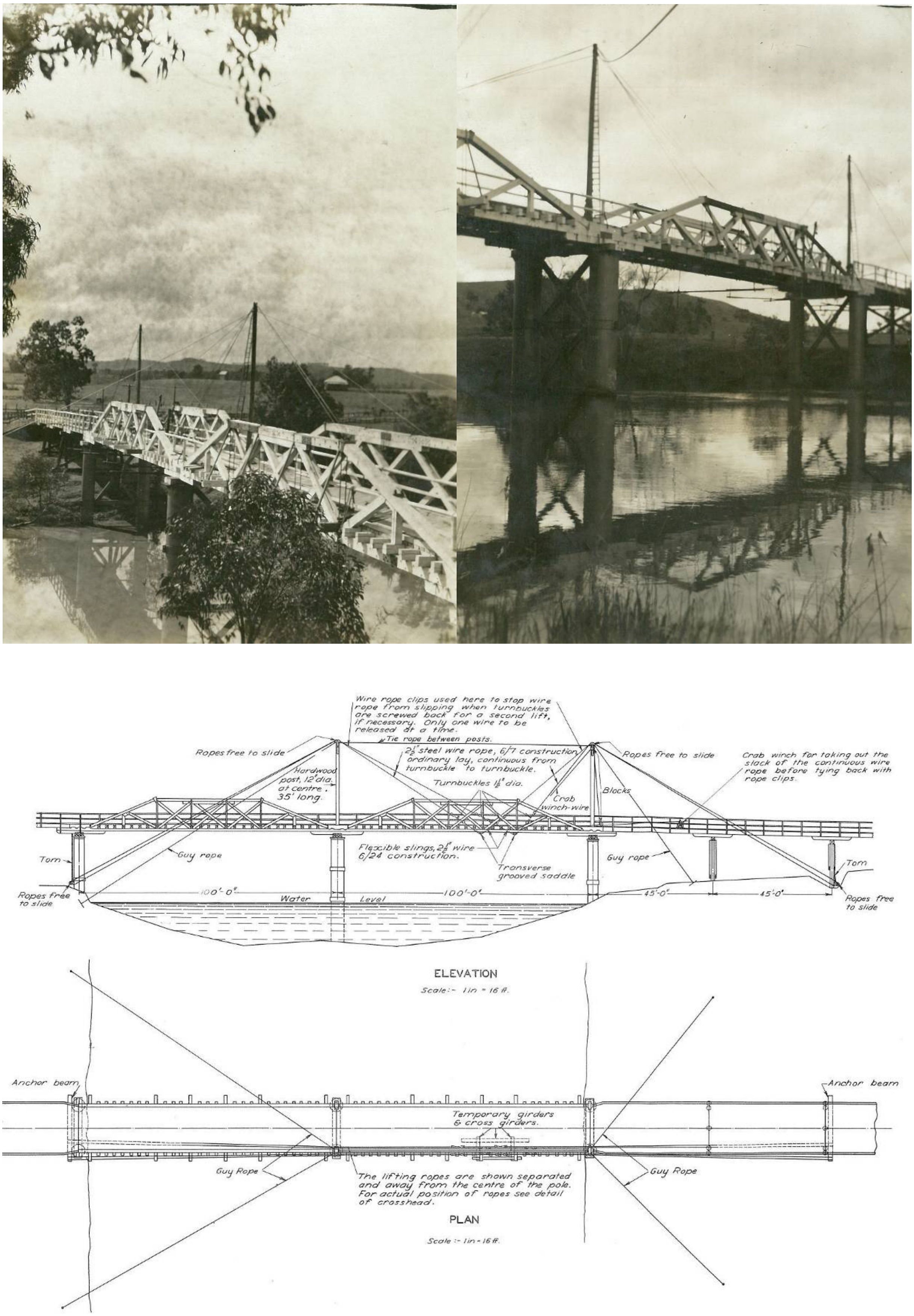


Figure 177: Clarence Town Bridge Temporary Support System in 1943 - Photographs and Plan

Longitudinal sheeting was added to both the truss and approach spans in 1959. Although the sheeting was only 50 mm thick, due to the large area of the deck, this would have added up to eight tonnes per truss span. Since that time, the thickness of sheeting has increased to 75 mm.

Again, despite significant member replacements with all new timber in 1943, less than 20 years later the sagging of the spans (both top and bottom chords) had become visually apparent, and the truss spans were under-trussed in 1962 (Figure 178). Under-trussing was normally a temporary measure in order to allow traffic to cross while the new (generally concrete) bridge was being designed and constructed. However, at Clarence Town, the new bridge did not eventuate. Despite the use of under-trussing the sag could not be removed from the truss spans at that time.

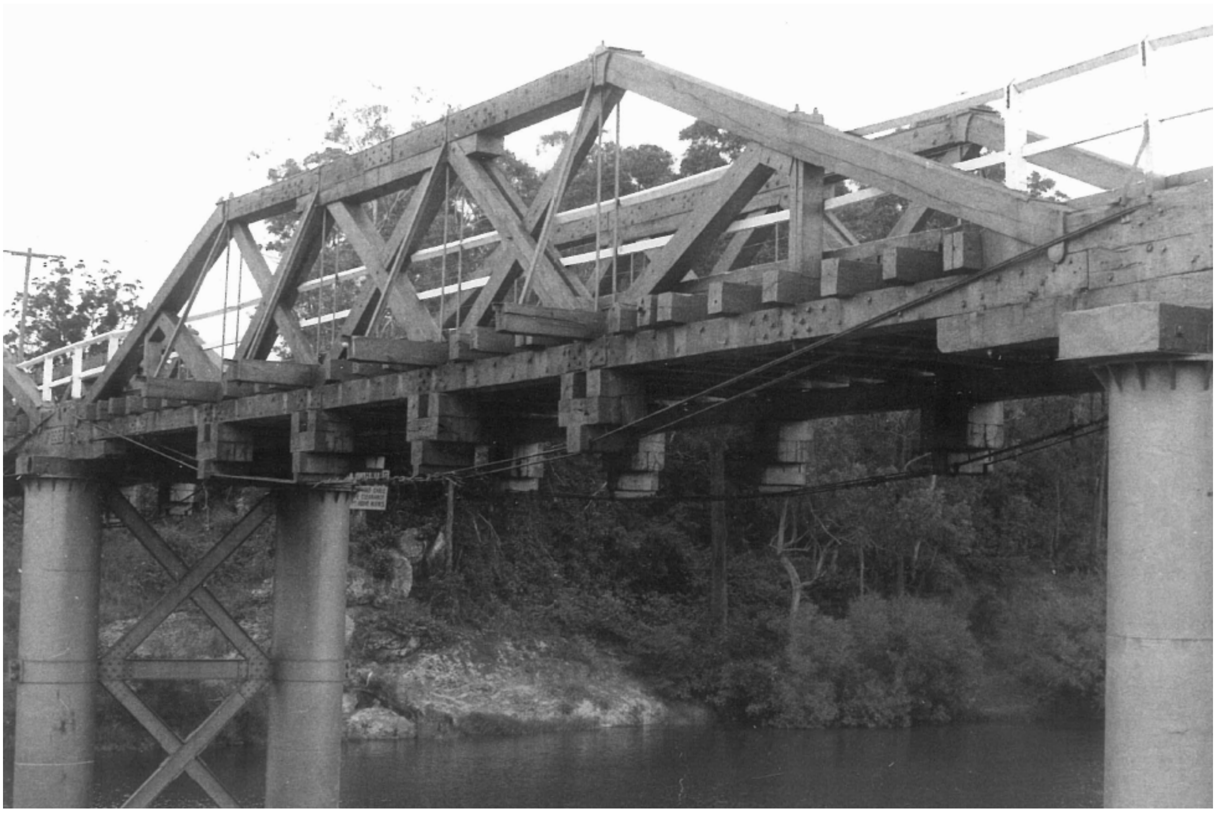


Figure 178: Clarence Town Bridge with Under-trussing (photograph taken in 1974)

In 1975, following the installation of another new principal (this time using a Bailey Bridge and a small crane – see Figure 179), restoration of camber and tightening of under-trussing, one of the cast iron shoes between the top chord and principal fractured as a 23 tonne crane passed over the bridge (Figure 180). An almost identical fracture had occurred approximately 15 years beforehand on the other truss span (Span 2). Two of the cast iron shoes at the base of the principals were also found to have been fractured for some time when inspected in 1975. The damaged bottom chord shoes were therefore replaced with steel shoes, as were the damaged top chord shoes. In addition to this, in order to prevent truss instability in the event of future sudden fracture of the other cast iron top chord shoes, additional tension rods were installed approximately 800 mm from the panel point, holding the principal to the bottom chord. This had been done for one principal in the early 1960s, and had been found to be very effective.

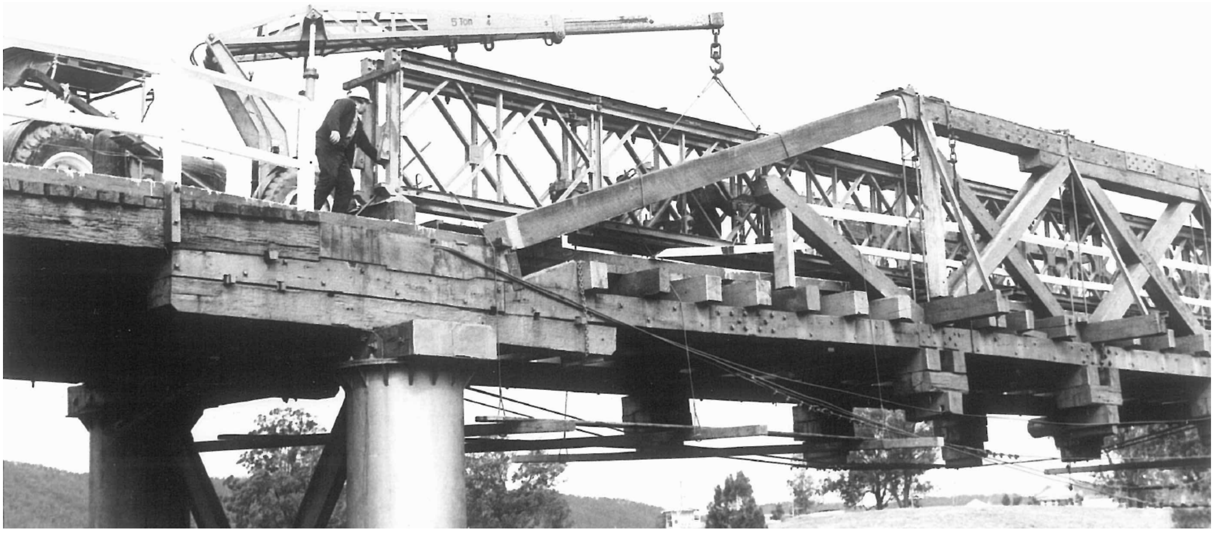


Figure 179: Replacement of Principal in 1975 using a Bailey Bridge (note under-trussing present too)

Figure 180: Top: Shoe Broken in 1975; Left: Original Shoe Remaining; Right: Steel Shoe Installed 1975

In addition to this, the wrought iron washer plates, originally detailed to slide into the top chord shoe and also support the tension rods just next to the shoe, have been removed, and separate smaller washer plates have been provided for the tension rods completely separate from the shoe. This is a sensible change, based at least in part, on the recommendations of the 1962 Bridge Maintenance Manual, which details issues with the original shoes (see Section 4.4.4 for details). Another sensible and necessary change was the replacement of the original wrought iron tension rods with new steel tension rods, able to take the heavier loads with less risk of sudden fracture.

During the 1990s, attempts were made to strengthen the bottom chord by increasing the section size of the timber and modifying the connections, doing away with the fish plates and instead providing splice plates with shear keys, similar to those used in Allan trusses. Unfortunately, since the original drawings were very difficult to read, existing timber lengths and joint locations were used, which were considerably shorter than the original. This meant that despite the significant increase in size of the bottom chord, the strength was not substantially improved because the length rather than the cross section size is the critical parameter. Also, instead of the new splice plates providing additional strength to the bottom chord, they instead had the effect of introducing additional eccentricities in the load paths (since the splice plates were only on one side of a very large timber bottom chord (originally 405 mm wide, now 510 mm wide) which is very different to an Allan truss where the splice plates are provided on both sides of a single flitch. This caused lateral deflections in the bottom chord as soon as load (self weight) was applied. As well as being ineffective in strengthening the bottom chord, these changes substantially affected the aesthetics of the truss. No longer were the top chords, principals and bottom chords all of the same cross section, thereby framing the truss, with the shoes clearly visible. Now the bottom chord was substantially larger than the other members, which obscured views to the tear-drop castings at the base of the principals. Moreover, instead of the original irregular layout of small (approx 915 mm long) fish plates, there was now a regular and symmetrical arrangement of very long (approx 1650 mm) splice plates with welded shear keys.

All the timber cross girders (both primary and secondary) were replaced in the late 1990s, and at this time they were changed so that all cross girders were the same cross sectional dimensions (320 deep by 225 wide), rather than the original configuration where primary cross girders were 330 deep by 255 wide and secondary cross girders were 330 deep by 150 wide (much more slender). Not only were secondary cross girders substantially upsized, but also they were lengthened, so that instead of extending only a little past the trusses, they extend almost a metre. These modifications combine to add very significant dead load to the bridge, which the bridge cannot sustain. Both truss spans have been supported by a permanent Bailey since 2006.

Paint schemes for Clarence Town Bridge have changed from white (both in the original, and in the 1926-27 reconstruction, in which even the piers were painted white) to grey (in 1965 the trusses were grey but the piers were again painted white) to red, then called "medium brown" (the new colour scheme chosen in 1991), with metal components (excluding piers) generally painted black, and the piers painted grey. The handrails have consistently been painted white.

All this means that designing on the basis of like for like replacement would be a really bad idea!

6.3.2 Design Actions

In accordance the provisions of Section 5.1, the following are used to determine design actions:

Dead Load:
Timber (not SLT): 11kN/m^3; Load Factor = 1.4
Timber (SLT): 11kN/m^3; Load Factor = 1.2
Metal: 78.5kN/m^3; Load Factor = 1.1
Live Load:
T44; DLA = 1.25; Load Factor = 2

A three dimensional model of a single truss span is prepared. Timber diagonals are pinned at both ends in both directions, but principals are fixed at the base and pinned at the top (this is due to the substantial restraint provided by the tapered timber spacer and diagonal bolts at the base of the principals). Top chords and bottom chords are continuous. Tension rods are fixed at both ends (since they are inserted through holes in the top and bottom chords). Tension rods are defined as tension-only members and timber diagonals are compression-only members. One end of the span is fixed in position and the other is free to move longitudinally. Supports are pinned.

Since a stress laminated timber (SLT) deck is now a standard treatment for timber trusses, and since it is especially beneficial in the earlier truss types (see Section 4.4.2), the model includes an SLT deck rather than the diagonal decking and longitudinal sheeting. Also, since barriers are to be upgraded to standard steel "ordinance style", the dead weight from such a system is included along with steel cross girders which generally replace under-capacity timber primary cross girders. The distance between kerbs is, by necessity, reduced with the introduction of an upgraded barrier rail, and so the T44 design vehicle is offset 600 mm from the new rail to determine design actions. New steel sway braces are also included in the model (free to translate vertically at top).

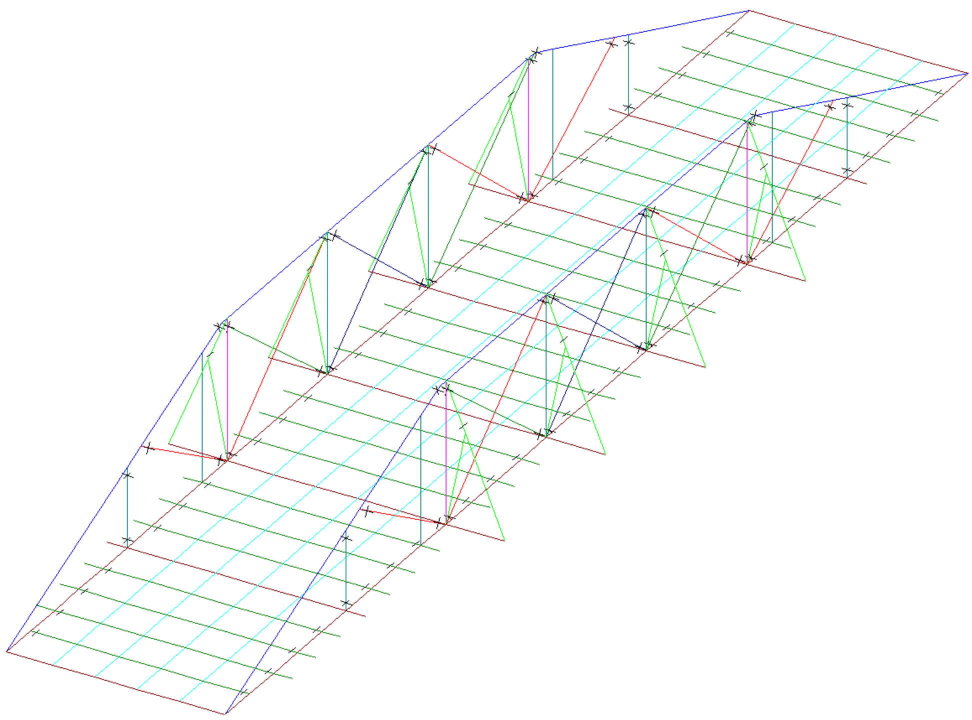


Figure 181: Three Dimensional Microstran Model for Clarence Town Bridge Rehabilitation Design

In order to accurately model the stiffness of a member in a timber truss, both the modulus of elasticity (E) and the second moment of area (I) must be given. For truss timber, the relevant modulus of elasticity is 16,000MPa (F22 timber). For most members in an Old PWD truss, the value for I_x can be simply calculated as $bd^3/12$. The only member where there may be some ambiguity is the double diagonal member which connects into the lower portion of the top chord. This is also the only member (other than the timber cross girders and the laminated timber bottom chord) in an Old PWD truss that tends to be under-capacity for modern loadings.

A lower-bound estimate of the stiffness of this double member could be found by doubling the value of I obtained for a single flitch. However, this is unlikely to provide sufficient stiffness, and will probably lead to member buckling and instability in the Microstran model. Since this member is laterally supported by a counterbrace, it is not unreasonable to include the stiffness of the counterbrace as well as the stiffness of the double member in the value of I_y. This is because there is a connection on the bridge which cannot be reflected in a Microstran Model. Diagonal members (counterbracing) are not connected in Microstran where they cross, otherwise instability would occur in the model due to excessive rigidity in the connection. The three flitches in question (the double and the single counterbrace) are generally of the same dimension, and so the value of I_y for the double member about the minor axis can be approximated by $I_y = 3db^3/12$. The value of I_x for the double member about the major axis, however, should be $I_x = 2bd^3/12$ as the third member does not provide significant restraint in this direction. For the particular case at Clarence Town, the double member also needs to be upsized, and so the relevant second moment of area about the minor axis is increased with the increased section size. If the model still has issues with instability and buckling, it is likely due to the fact that the effective length in the model is significantly longer than the actual effective length unless rigid offsets are provided. Therefore, in order to ensure that the model gives realistic results, a further modification of the I value to take into effect the effective length can be done ($I_{y,mod} = I_y / L_{actual}^2 \times L_{model}^2$).

This three dimensional model then provides the following axial and bending envelopes:

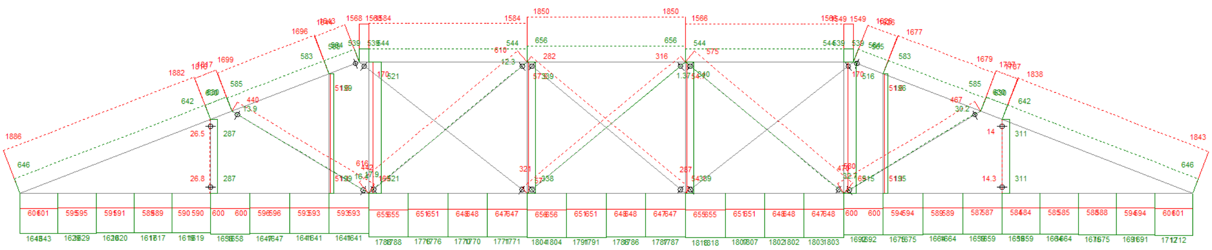


Figure 182: Design Actions (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

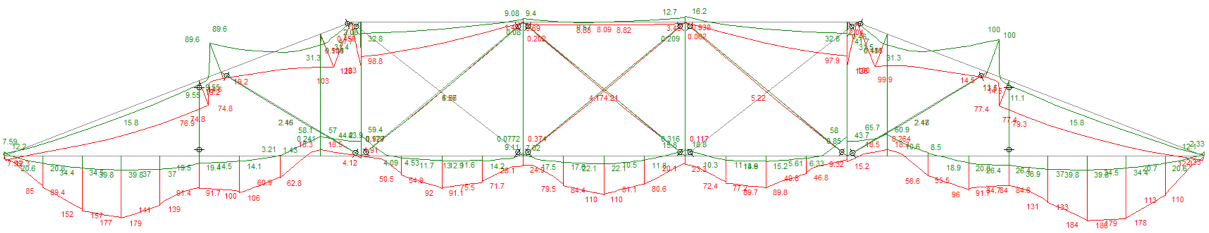


Figure 183: Design Action (Bending Moment Envelope): Ultimate Dead Load + Ultimate T44 Live Load

6.3.3 Calculating Capacities of Principals

The capacity in compression of a principal can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\ k_1\ f'_c\ A_c$; or
$= \phi\ N_{cr}$

N^*_c	=	design action effect in compression (1885 kN)
ϕ	=	capacity factor (0.75)
k_1	=	duration of load factor (0.97)
f'_c	=	characteristic value, compression parallel to grain (42 MPa)
A_c	=	cross-sectional area (406 x 356 = 144,536 mm^2)

$$N_{cr} = \frac{\pi^2 E_{0.05} I}{(g_{13} L)^2}$$

$E_{0.05}$	=	lower 5th percentile modulus of elasticity (8,000 MPa)
I_y	=	$db^3/12$ for a rectangular section (1,525 x 10^{-6}mm^4)
I_x	=	$bd^3/12$ for a rectangular section (1,985 x 10^{-6}mm^4)
g_{13}	=	effective length factor (0.7 about major axis, 0.5 about minor axis)
L	=	length of member in mm (8,077 mm)

Therefore, $N_{d,c}$ = 3675 kN

The capacity in bending of a principal can be calculated as follows:

$M_d \geq M^*$ where $M_d = \phi\ k_1\ k_{11}\ k_{21}\ f'_b\ Z$

M^*	=	design action effect in bending (M^*_x = 20 kNm; M^*_y = 100 kNm)
ϕ	=	capacity factor (0.75)
k_1	=	duration of load factor (0.97)
k_{11}	=	$(300/d)^{0.167}$ (major axis 0.95; minor axis 0.97)
f'_b	=	characteristic value in bending (55 MPa)
Z	=	$db^2/6$ for bending of about minor axis (8.58 x 10^{-6}mm^3)
	=	$bd^2/6$ for bending of about major axis (9.78 x 10^{-6}mm^3)

Therefore, $M_{d,x}$ = 300 kNm; $M_{d,y}$ = 265 kNm

We can therefore verify that the principals have sufficient capacity under biaxial bending and compression using the following conservative criteria provided in AS1720.1 Appendix E:

$$\left(\frac{M^*_y}{M_{d,y}}\right)^2 + \left(\frac{M^*_x}{M_{d,x}}\right) + \left(\frac{N^*_c}{N_{d,cy}}\right) = 0.72 \leq 1.0$$

$$\left(\frac{M^*_y}{M_{d,y}}\right) + \left(\frac{M^*_x}{M_{d,x}}\right)^2 + \left(\frac{N^*_c}{N_{d,cy}}\right) = 0.89 \leq 1.0$$

6.3.4 Calculating Capacities of Diagonals

The capacity in compression of the timber diagonals can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\, k_1\, f'_c\, A_c$; or
$= \phi\, N_{cr}$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor (0.97 for ultimate live loads)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a single flitch
N_{cr}	=	critical elastic buckling load ($\pi^2 E_{0.05} I/L^2$)

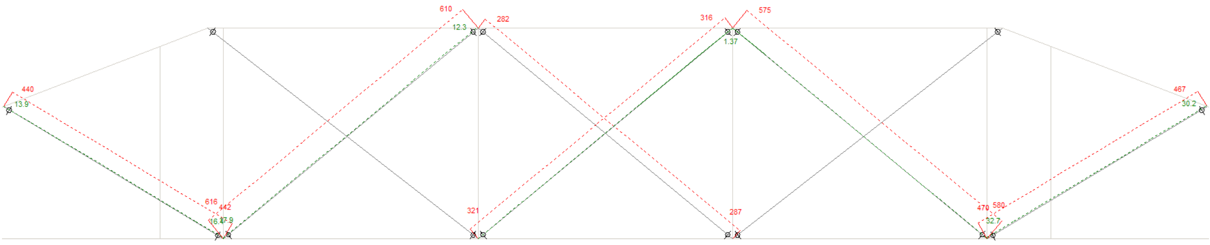


Figure 184: Design Action (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

Table 36: Compressive Capacities of Diagonals for Clarence Town Old PWD Truss

	N^*_c	$\phi\, N_{cr}$	$\phi\, k_1\, f'_c\, A_c$	
Short 305 x 254 x 3962 mm	470	1570	2365	OK
Double 2/305 x 152 x 4877mm	620	445	2830	Fail
Central 305 x 305 x 4877 mm	320	1795	1415	OK
Counter 305 x 152 x 5182 mm	0	195	1415	OK

6.3.5 Recommendations for Timber Members

Therefore, changes from original details to meet current design loads are limited to the following:

- Increase size of flitches in double diagonals from 305 x 152 mm to 305 x 175 mm
- Provide either internal steel box or external steel plates to strengthen bottom chord
- Modify connections of counterbracing to allow for thermal and shrinkage movements
- Provide SLT deck with connections detailed to ensure lateral stiffness
- Replace timber primary cross girders with steel of hollow rectangular section
- Provide appropriately strengthened steel sway bracing instead of original timber (Figure 173)

6.4 A McDonald Truss – McKanes Bridge

6.4.1 Heritage Significance

Figure 185: McKanes Bridge over the Cox's River near Lithgow

McKanes Bridge (built 1893) crosses the Cox's River with two 90' McDonald trusses (the longest span constructed of this type) founded on the original masonry abutments with a central concrete pier constructed in the 1980s. The original masonry pier suffered severe flood damage in 1986, so that it rotated and cracked and moved downstream by approximately 1 m. As the photographs on the next page show, the trusses survived the distortion amazingly well. Other changes from the original configuration at McKanes include the removal of the original diagonal decking and its replacement with longitudinal decking with a spray seal wearing surface. This necessitated the addition of various timber elements below the deck to hold the deck down. Also, the original timber rails without posts on the trusses and without end posts (shown in Figure 186) have been replaced with a barrier system typical of the later timber truss types. The original metal bottom chord cover plates have also been removed with no replacement provided.

The bridge carries a single lane of traffic, being only 4.57 m wide between kerbs. A plaque has been installed next to the bridge as a Historic Engineering Marker by the Institution of Engineers Australia because of its significance as a representative of McDonald truss road bridges.

Figure 186: Flood Damage to Central Pier of McKanes Bridge over the Cox's River near Lithgow

6.4.2 Design Actions

In accordance the provisions of Section 5.1, the following are used to determine design actions:

Dead Load:
Timber (not SLT): 11kN/m^3; Load Factor = 1.4
Timber (SLT): 11kN/m^3; Load Factor = 1.2
Metal: 78.5kN/m^3; Load Factor = 1.1
Live Load:
T44; DLA = 1.25; Load Factor = 2

A three dimensional model of a single truss span is prepared. Timber diagonals are pinned at both ends in both directions, but principals are fixed at the base and pinned at the top (this is due to the substantial restraint provided by the diagonal bolts and secondary cross girders). Top and bottom chords are continuous. Single tension rods are fixed at both ends (since they are inserted through holes in the top and bottom chords). Double tension rods are pinned at both ends. Tension rods are defined as tension-only members and diagonals are compression-only members. One end of the span is fixed in position (pinned) and the other is free to move longitudinally.

Since a stress laminated timber (SLT) deck is now a standard treatment for timber trusses, and since it is especially beneficial in the earlier truss types (see Section 4.4.2), the model includes an SLT deck rather than the diagonal decking and longitudinal sheeting. Also, since barriers are to be upgraded to standard steel "ordinance style", the dead weight from such a system is included along with steel cross girders which generally replace under-capacity timber primary cross girders.

Dead load from steel "ordnance style" barriers can be up to 125kg/m.

The distance between kerbs is, by necessity, reduced with the introduction of an upgraded barrier rail, and so the T44 design vehicle is offset 600 mm from the new kerb to determine design actions. New steel sway braces are also included in the model (free to translate vertically at top).

One of the differences between an Old PWD truss and a McDonald truss is that in a McDonald truss, all the primary elements of the truss (chords, diagonals, tension rods) line up without any eccentricity at the nodes. In order to achieve this, McDonald introduced timber spacers next to some of the primary cross girders, which in turn means that some secondary cross girders have significant notches cut out of the tops of them in order to accommodate the diagonals. This would cause significant difficulties if the secondary cross girders needed to be upgraded to steel.

The stiffness of the double diagonals can be determined by an elastic critical buckling analysis of the two member assembly with the counterbrace also included. Further adjustment to the second moment of area can be calculated to take into account the difference between the length of the member in the model (node to node) and the actual length ($I_{y,mod} = I_y / L_{actual}^2 \times L_{model}^2$).

The bottom chord generally requires replacement with a new steel bottom chord, which has successfully been achieved in the past with plates, for the 90' span, 2/32mm plates are required.

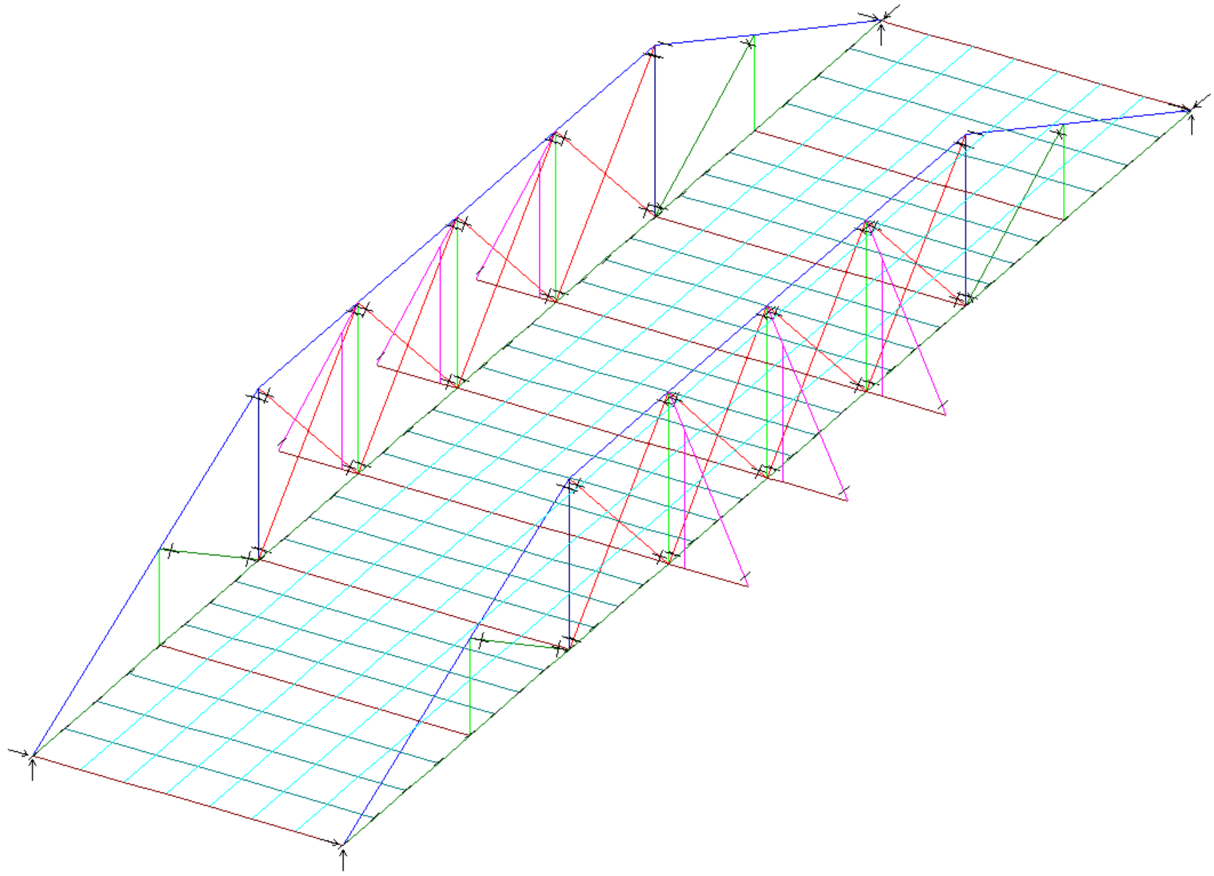


Figure 187: Three Dimensional Microstran Model for McKanes Bridge Rehabilitation Design

This three dimensional model then provides the following axial and bending envelopes:

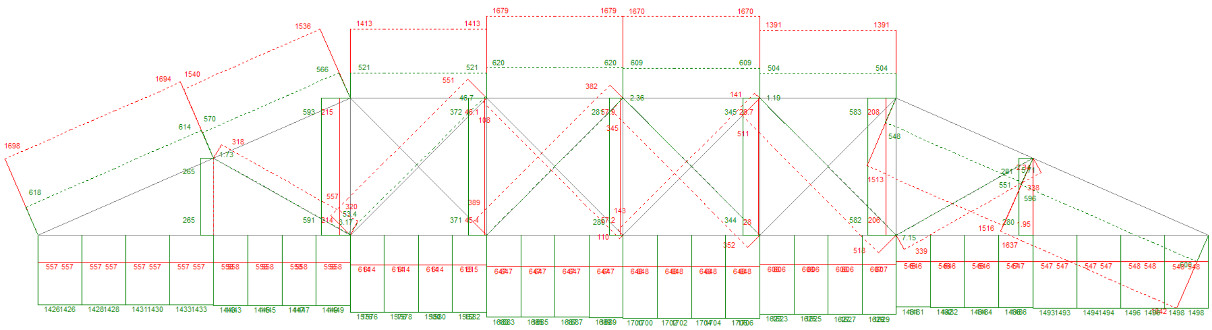


Figure 188: Design Actions (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

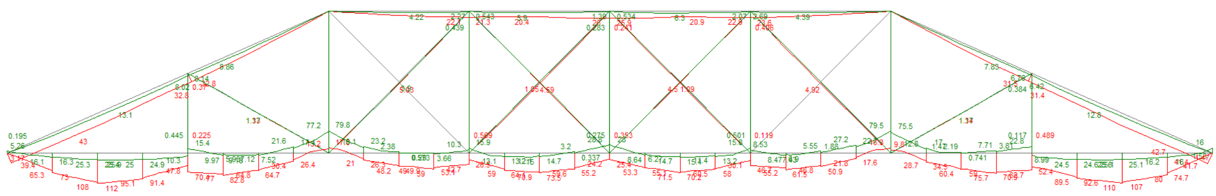


Figure 189: Design Action (Bending Moment Envelope): Ultimate Dead Load + Ultimate T44 Live Load

6.4.3 Calculating Capacities of Principals

In the past, attempts have been made to improve the lateral stability of the tops chords of McDonald trusses by providing additional sway braces at the tops of principals. Unfortunately, the geometry of the McDonald truss makes this awkward, as the tension rods are external to the top chord, not allowing sway braces to be connected, so either the sway braces are put on an angle (as was done at Five Day Creek) or are connected eccentric to the cross girder (as was done at Junction Bridge over the Tumut River) as shown in the photographs below. This is quite different to the original design intent and gives an awkward look to the bridge, whereby the splayed principals were always intended to provide sufficient lateral stability for the entire span.

Figure 190: Additional Sway Bracing (top) Five Day Creek, (bottom) Junction Bridge Tumut River

Indeed, if the original detail of the solid timber top chord is provided, and the original solid timber spacers in the principal, then the principal gives sufficient lateral stability even under T44 loading, and the additional sway brace is not required. Unfortunately, these large and long section timbers are increasingly difficult to obtain. For example, at Junction Bridge, the solid timber top chord was replaced with two timbers bolted together, which has only a fraction of the lateral stiffness of the original member, and therefore necessitated additional lateral support.

In order to calculate the capacity of the principal, an elastic critical buckling analysis can be undertaken. The supports are modelled as pinned at the base and free at the top. However, connected to the top is the first panel of the top chord (3.2 m in length), which is pinned at the end where it meets the principal, but fixed in both position and direction at the other end (free to translate in the vertical plane so that the model applies the appropriate compression to the principal). This model simulates the restraint provided by the cantilevered section of the top chord. Also, the structural spacers are here included in the model, but only a small fraction of the actual area is modelled (same area as the flitches) and the spacers are pinned at one end and fixed at the other, representing the looseness of the connection on one side as geometry changes due to buckling, but the robustness of the notched connection on the other side.

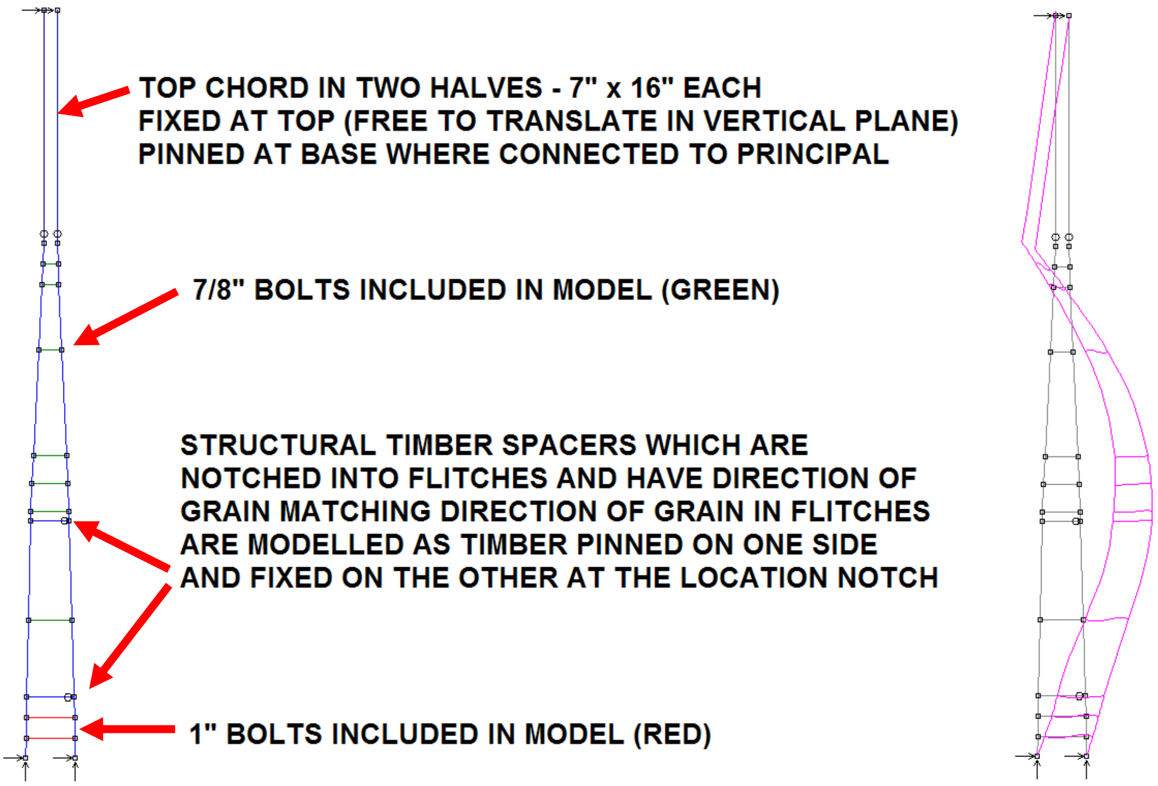


Figure 191: Critical Elastic Buckling Analysis of Principal in McDonald Truss

The critical elastic buckling analysis gives a nominal capacity of 2856 kN, which means the design capacity is 2140 kN, which is significantly greater than the design compressive force of 1700 kN. Bending stresses (due to fabrication) can be minimised to ensure that the principal is not overstressed in combined bending and compression by careful design of spacer dimensions to limit the curvature in the flitches (since actual curvature is not specified in the original drawings).

6.4.4 *Calculating Capacities of the Top Chord*

The capacity of the top chord can also be determined by an elastic critical buckling analysis (Figure 192). A model is prepared of the top chord in plan view. The model is fixed in position at the centre, and fixed in the lateral direction only at the other two sway brace locations. The ends are completely free. The relevant loads from the global model are applied at the nodes. The modulus of elasticity of the timber is the lower fifth percentile value of 8,000 MPa.

This gives a design capacity of 1560 kN for the outer panels, and 1850 kN for the internal panels. The single solid top chord therefore has sufficient capacity with no need for extra sway braces.

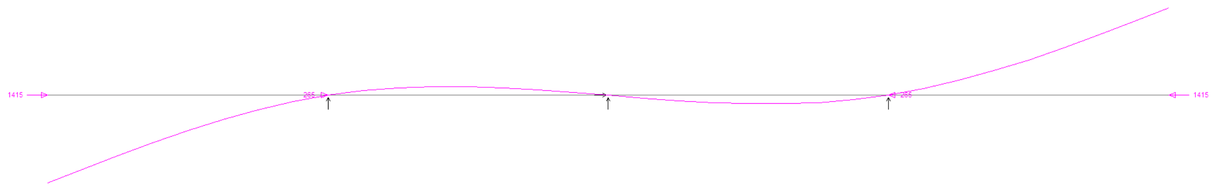


Figure 192: Critical Elastic Buckling Analysis of Top Chord in a 90' McDonald Truss

6.4.5 *Calculating Capacities of Diagonals*

The capacity of the double diagonals can be determined using the same model as was used to determine an equivalent stiffness in Section 6.4.2 for the global model, but the modulus of elasticity for the timber must be reduced to the lower fifth percentile value of 8,000 MPa.

This gives a design capacity of 825 kN and is sufficient for combined bending and compression.

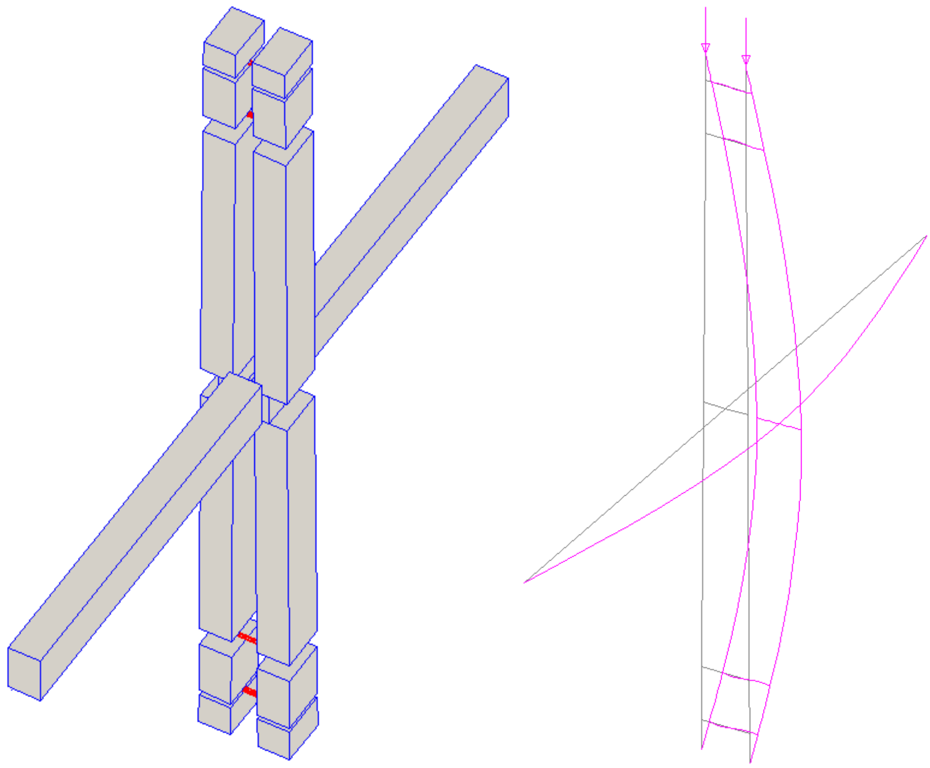


Figure 193: Critical Elastic Buckling Analysis of Double Diagonals in a McDonald Truss

6.4.6 Options for the Bottom Chord

A feature of the McDonald truss which does not occur in any other truss type is that the bottom chord is notched to accommodate the cross girders (both primary and secondary). In the Old PWD trusses, the cross girders were notched around the bottom chord (thereby assisting to keep the bottom chord in a straight alignment), but in the McDonald trusses, the notching into the bottom chord allows the forces from the diagonals to be carried into the bottom chord via the cross girders. This flow of forces is important because of the adjustable wedges, which do not allow forces to be transferred directly from the diagonals to the bottom chord because a gap opens when the wedges are tightened. The mortise and tenon joints are only provided to restrain the diagonals from lateral movement, not to transfer the axial loads from the diagonals.

For this reason also, the wedges are not placed in the notch, but slide above the bottom chord.

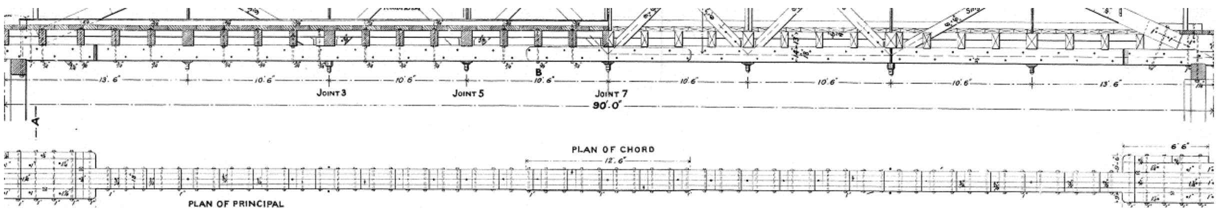


Figure 194: Notching of Bottom Chord in McDonald Truss – Plan for Standard 90' Span

Considerable care was taken by McDonald to maximise the cross sectional area of load bearing timber in the bottom chord. The connections of secondary cross girders are an example, whereby butt joints match with secondary cross girder locations so that vertical bolts go through the butt joint and thereby minimise losses in the bottom chord. If there is no butt joint, the vertical bolt is located through the centre of the bottom chord (between two laminates), and if there is a butt joint, the bolt is moved to the left or the right to pass through the butt joint.

Since secondary cross girders are connected in the gaps between laminates, the bolts have negligible shear strength, but just prevent the cross girders from lifting. The shear strength in the longitudinal direction is provided by the cross girders being notched into the bottom chord.

The longest length of timber used in a McDonald truss is the 16.3m length used at the ends of the bottom chords for the 90' spans (four of these lengths would have been originally installed at McKanes). Even by the end of the 1880s, however, this length of timber was very difficult to obtain, especially since it had to be the best quality heart free sap free timber. Therefore, when elements of the bottom chords have been replaced over subsequent years, they have been replaced with shorter members. This, in turn, reduces the total capacity of the bottom chord.

Another problem with the laminated timber bottom chord is that it is not as durable as, say, the Allan truss bottom chord where the two flitches are spaced. The laminated timber bottom chord tends to hold moisture, and also tends to attract hidden deterioration because of the large number of timber to timber interfaces, where moisture can get trapped and termites can hide.

Because of the reduced capacity from smaller lengths of timber and also because of the risks of hidden deterioration, some McDonald trusses, such as those at Bombala and Galston Gorge, have had steel plates added to the inner and outer faces of the bottom chord. Since these plates have been painted white, they are hardly detectable except upon close inspection, but if properly detailed, can give substantial increase in strength and robustness to a McDonald truss bridge.

Figure 195: External Steel Plates used to Strengthen Bottom Chord of Galston Gorge Bridge

This strengthening by external steel plates is more suitable for McDonald trusses than strengthening with an internal box section, which may be applicable for Old PWD trusses. The largest force to be transferred into the bottom chord is the force from the principals, and since the principals are splayed, they apply the force to the outer laminates of the bottom chord. Also, the stability of the top chord is dependent upon the splayed principals being firmly supported at the base, and a layer of timber cladding is not sufficient to provide the support required.

To take the required forces in a 90’ McDonald truss, two 32 mm thick steel plates are required for each bottom chord. These can have 26 mm deep notches to fit the primary cross girders, just as the original timber bottom chords were notched approximately 1” to accommodate the cross girders and also to transfer horizontal forces from the diagonals to the bottom chord. The secondary timber cross girders should be notched over the bottom chord so that the timber bears on timber (as it did originally), rather than on steel, which could cause localised crushing.

Continuity of the bottom chord over the central pier is critical to the design, as the load from the principal is transferred to the bottom chord in the adjacent span through a notch in the butting block. If continuity is lost, so is the load path from the principal to the bottom chord. For this reason, it is sensible to make both truss spans fixed at the centre pier, and allow thermal expansion at the abutments. It is also preferable to construct the two spans together.

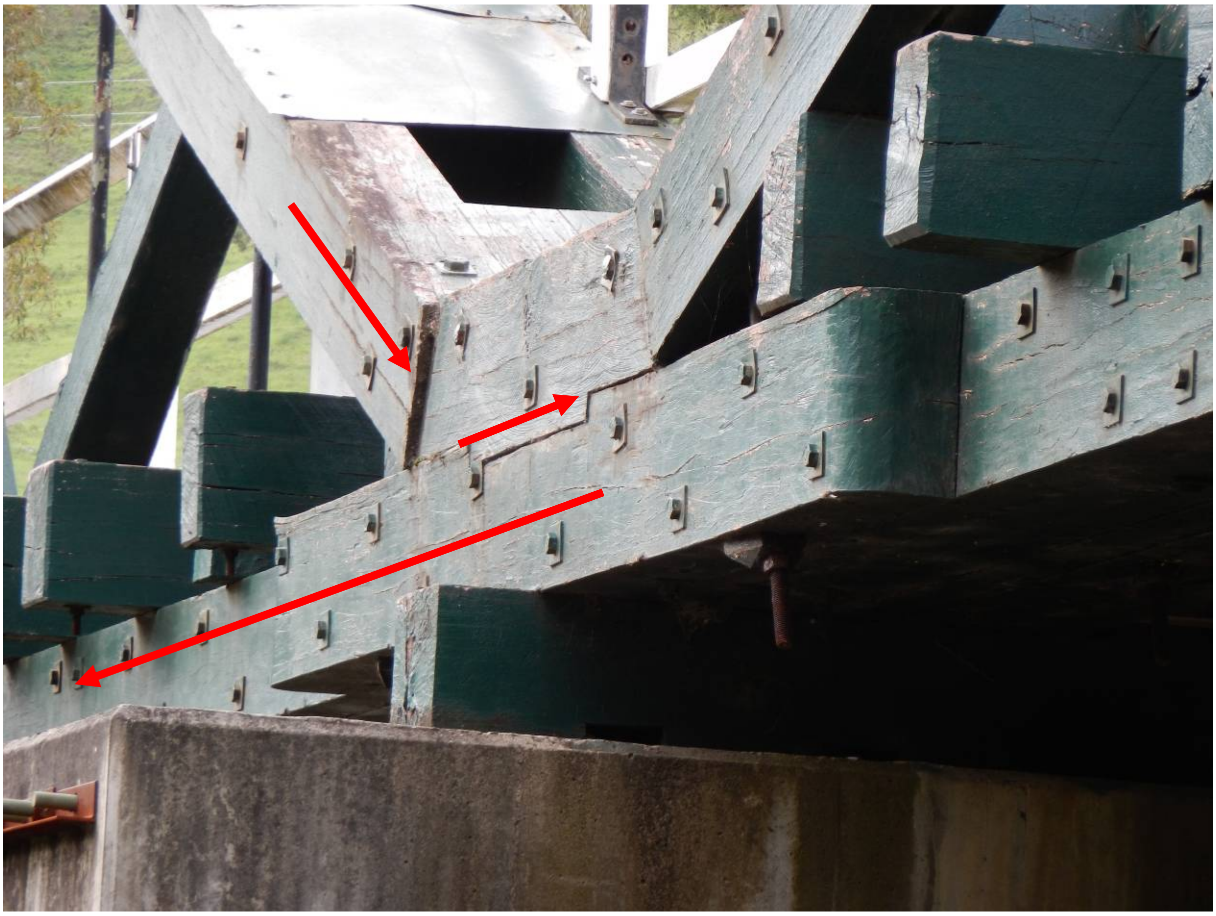


Figure 196: Load Path from Principals to Bottom Chord in Adjacent Span through Butting Block

6.4.7 Recommendations for 90' McDonald Truss Strengthening

Therefore, changes from original details to meet current design loads are limited to the following:

- Provide external steel plates to bottom chord notched to fit steel primary cross girders
- Replace timber barriers with steel barriers to increase road safety and protect the bridge
- Modify connections of counterbracing to allow for thermal and shrinkage movements
- Provide SLT deck with connections detailed to ensure lateral stiffness and take barrier loads
- Replace timber primary cross girders with steel of hollow rectangular section
- Provide appropriately strengthened steel sway bracing with additional knee braces
- Replace brittle grey cast iron shoes with new replica ductile cast iron shoes

6.5 Another Old PWD Truss – Monkerai Bridge

6.5.1 Heritage Significance

Figure 197: Monkerai Bridge over the Karuah River, as constructed in 1882

Monkerai Bridge over the Karuah River is the second oldest remaining timber truss bridge in NSW, and consists of three 70' Old PWD trusses at one end and three timber girder approach spans at the other end, all on single timber trestle piers. The bridge has undergone changes throughout its life, but many features of the bridge are still largely original. For example, the original cast iron shoes with wrought iron sliding washer plates are still in place (although some are fractured in areas), and the original detail of sawn outer timber girders providing continuity from the truss butting blocks back to the abutments through the approach spans still remains.

There have been only two significant changes which have negatively affected the capacity of this bridge. One has been the modification of the diagonal decking by the removal of the spiking planks, the introduction of spaces between the planks and the addition of longitudinal sheeting and timber kerbs along the length of the bridge (which reduces the lateral stiffness of the deck as well as adding significant dead load). The other has been the replacement of the original timber sway bracing with steel sway bracing at a lower angle on lengthened cross girders (this is ineffective, and also adds to the dead load of the bridge due to the lengthened cross girders).

Unlike Clarence Town, the sizes of the timber members are largely as original and still obtainable.

6.5.2 Design Actions

In accordance the provisions of Section 5.1, the following are used to determine design actions:

> **Dead Load:**
> Timber (not SLT): 11kN/m^3; Load Factor = 1.4
> Timber (SLT): 11kN/m^3; Load Factor = 1.2
> Metal: 78.5kN/m^3; Load Factor = 1.1
> **Live Load:**
> T44; DLA = 1.25; Load Factor = 2

A three dimensional model of a single truss span is prepared using the same assumptions that were used for Clarence Town, the only other remaining Old PWD truss (see Section 6.3.2).

Since a stress laminated timber (SLT) deck is now a standard treatment for timber trusses, and since it is especially beneficial in the earlier truss types (see Section 4.4.2), the model includes an SLT deck rather than the diagonal decking and longitudinal sheeting. Also, since barriers are to be upgraded to standard steel "ordinance style", the dead weight from such a system is included along with steel cross girders which generally replace under-capacity timber primary cross girders. The distance between kerbs is, by necessity, reduced with the introduction of an upgraded barrier rail, and so the T44 design vehicle is offset 600 mm from the new rail to determine design actions. New steel sway braces are also included in the model (free to translate vertically at top).

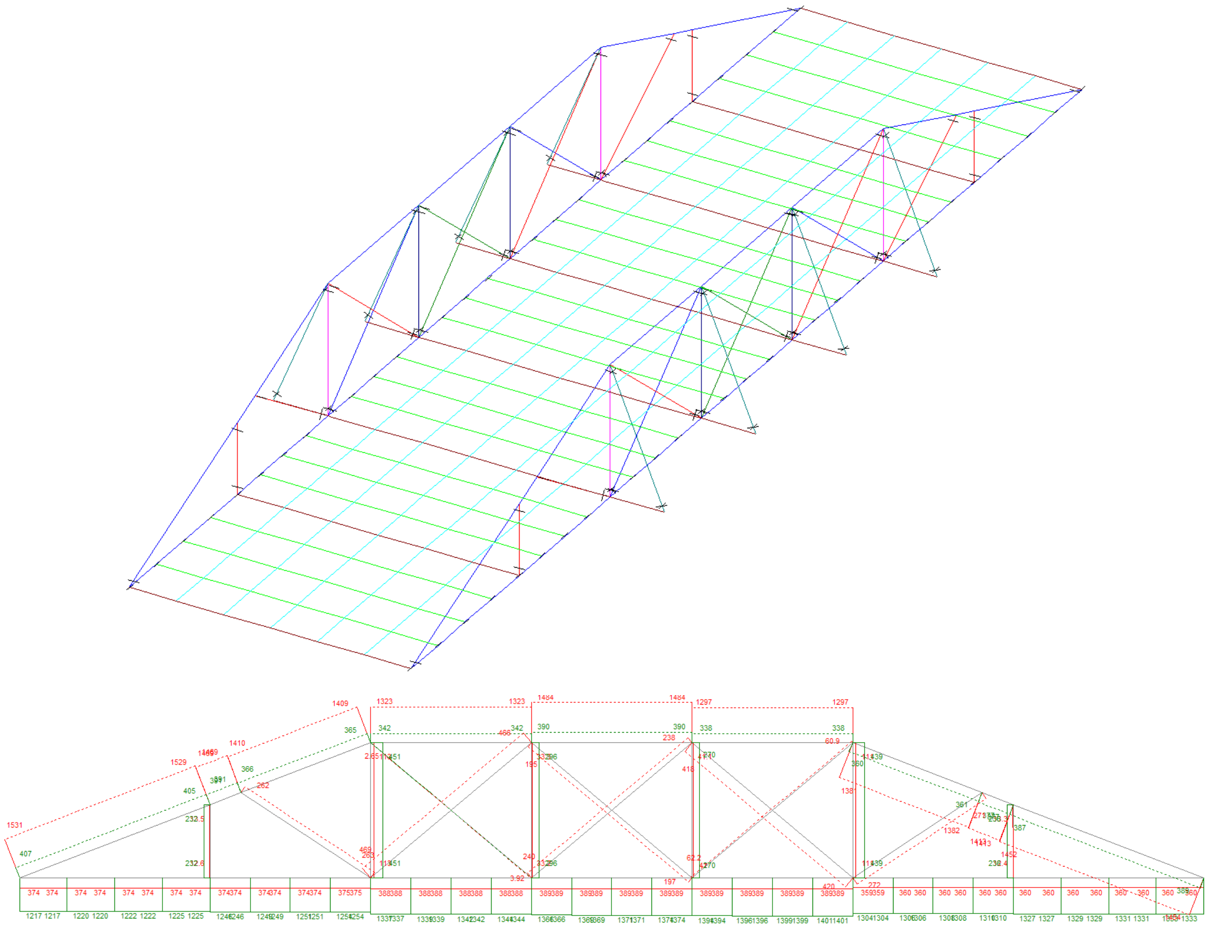


Figure 198: Three Dimensional Modal and Resulting Design Actions (Axial Envelope)

6.5.3 Calculating Capacities of Principals

The capacity in compression of a principal can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$= \phi\ k_1\ f'_c\ A_c$; or
$= \phi\ N_{cr}$

N^*_c	=	design action effect in compression (1530 kN)
ϕ	=	capacity factor (0.75)
k_1	=	duration of load factor (0.97)
f'_c	=	characteristic value, compression parallel to grain (42 MPa)
A_c	=	cross-sectional area (304 x 304 = 92,416 mm^2)

$$N_{cr} = \frac{\pi^2 E_{0.05} I}{(g_{13} L)^2}$$

$E_{0.05}$	=	lower 5th percentile modulus of elasticity (8,000 MPa)
I	=	$db^3/12$ for a rectangular section (712 x 10^{-6}mm^4)
g_{13}	=	effective length factor (0.7 about major axis, 0.5 about minor axis)
L	=	length of member in mm (5,865 mm)

Therefore, $N_{d,c}$ = 2500 kN (buckling failure) and $\phi\ N_{cr}$ = 2825 kN (material failure)

The capacity in bending of a principal can be calculated as follows:

$M_d \geq M^*$ where $M_d = \phi\ k_1\ k_{11}\ k_{21}\ f'_b\ Z$

M^*	=	design action effect in bending (M^*_x = 5 kNm; M^*_y = 90 kNm)
ϕ	=	capacity factor (0.75)
k_1	=	duration of load factor (0.97)
k_{11}	=	$(300/d)^{0.167}$ (= 0.998)
f'_b	=	characteristic value in bending (55 MPa)
Z	=	$db^2/6$ (= 4.68 x 10^{-6}mm^3)

Therefore, $M_{d,x}$ = 185 kNm; $M_{d,y}$ = 185 kNm

We can therefore check to see if the principals have sufficient capacity under biaxial bending and compression using the following conservative criteria provided in AS1720.1 Appendix E:

$$\left(\frac{M^*_y}{M_{d,y}}\right)^2 + \left(\frac{M^*_x}{M_{d,x}}\right) + \left(\frac{N^*_c}{N_{d,cy}}\right) = 1.09 > 1.0$$

$$\left(\frac{M^*_y}{M_{d,y}}\right) + \left(\frac{M^*_x}{M_{d,x}}\right)^2 + \left(\frac{N^*_c}{N_{d,cy}}\right) = 0.87 \leq 1.0$$

We see that the principal fails under these simplified criteria by less than 10%, but if we consider more closely the failure mode, the critical combination is bending about the axis which is not subject to buckling effects, so if the material strength is considered rather than the buckling strength then the result becomes closer to 1.0 and so strengthening is not really necessary.

Even though theoretically this member may be slightly under-capacity, the risk is low. On the other hand, if attempts were made to strengthen this member, the heritage impact would be high.

One way to theoretically provide additional capacity would be to provide an additional sway brace half way up the principal to restrain it from buckling sideways, as has been suggested for some Old PWD and McDonald trusses from time to time. Unfortunately, this would have a significant visual impact on the bridge, and does not actually achieve the desired capacity at Monkerai because the failure is primarily due to bending stresses, not compressive buckling.

Another way to provide additional capacity is to upsize the principal. Unfortunately this would have a detrimental effect on the heritage of the bridge due to the fact that one of the defining features of the Old PWD is the use of the same size for top chord, bottom chord and principals.

6.5.4 Calculating Capacities of Diagonals

The capacity in compression of the timber diagonals can be calculated as follows:

$N_{d,c} \geq N^*_c$ where $N_{d,c}$ is the lesser of:

$$= \phi\, k_1\, f'_c\, A_c; \text{ or}$$
$$= \phi\, N_{cr}$$

$N_{d,c}$	=	design capacity in compression
N^*_c	=	design action effect in compression
ϕ	=	capacity factor (0.75 for F22 timber)
k_1	=	duration of load factor (0.97 for ultimate live loads)
f'_c	=	characteristic value in compression parallel to grain (42MPa for F22)
A_c	=	cross-sectional area of a single flitch
N_{cr}	=	critical elastic buckling load ($\pi^2 E_{0.05} I/L^2$)

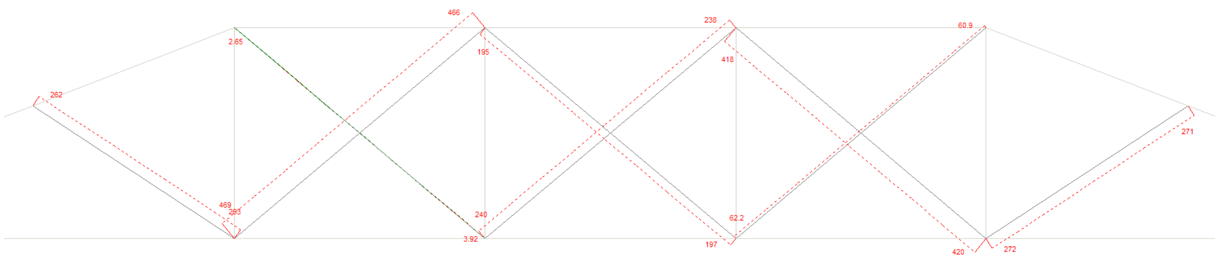


Figure 199: Design Action (Axial Envelope): Ultimate Dead Load + Ultimate T44 Live Load

Table 37: Compressive Capacities of Diagonals for Monkerai Bridge Old PWD Truss

	N^*_c	ϕN_{cr}	$\phi k_1 f'_c A_c$	
Short 230 x 202 x 2600 mm	275	1385	1420	OK
Double 2/230 x 152 x 3510mm	470	650	2135	OK
Central 230 x 230 x 3365 mm	240	1220	1615	OK
Counter 230 x 152 x 3510 mm	60	325	645	OK

6.5.5 Differences between the 100' and the 70' Old PWD Truss

There are some key differences between the 100' and the 70' Old PWD trusses which influenced their original design and therefore must be considered in any rehabilitation or strengthening.

6.5.5.1 Detailing of Double Diagonals and Top Chord

One difference is the length and detailing of the lower portion of the top chord.

Longer spans have the double top chord extending beyond the panel points (Fig 25), whereas shorter spans have the double top chord stopping neatly at the panel points (Fig 26). This is because, for shorter spans, the double diagonal consists of members exactly half the width of the top chord bolted together with no gap, bearing directly against the end of the bottom top chord.

This obviously means that the original lengths should be used based on the original drawings for that span length rather than scaled down from a different span length, but more importantly it means that the double diagonals cannot be upsized in the shorter Old PWD trusses without sticking out further than the top chord and principals and have a deleterious impact on aesthetics.

6.5.5.2 Availability of Timber for Laminated Timber Bottom Chord

Another difference is the size and availability of timber, especially for the bottom chord.

As can be seen in Figure 112, for Monkerai Bridge only eight pieces of timber in the bottom chord for the whole bridge (3 spans) are longer than 10m, and the cross sectional dimensions are limited to only 304 x 102 mm. Though expensive, this timber is still procurable today.

In contrast, for Clarence Town Bridge, almost all the timber in the bottom chord is well in excess of 10m in length and one piece is in excess of 18m. The timber has larger cross sectional area of 356 x 127 mm and is not available today, and so the original design simply cannot be achieved. It is therefore possible to retain an original design feature at Monkerai which it is simply not possible to retain at Clarence Town, which is the laminate layout for the bottom chord.

Of the two structurally viable options for strengthening Old PWD truss bottom chords (ie. internal steel box or external steel plates) the external steel plate option retains the laminate layout (Fig 201). This option also minimises other changes by allowing the original connection details to be used between the timber diagonals and the laminated timber bottom chord (ie. mortise and tenon joints) rather than having to introduce large steel plates and bracket connections with additional bolts as is necessary for the internal steel box strengthening option.

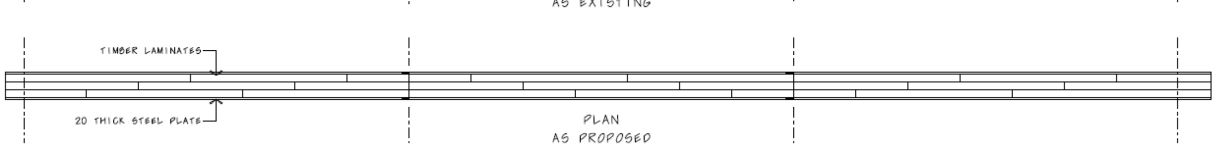


Figure 200: Strengthening of Bottom Chord while Retaining Original Laminate Layout

6.5.5.3 Locations of Tension Rods and Influence on Bottom Chord Strengthening

Another difference which has implications for bottom chord is found in the tension rods.

While the longer spans (such as Clarence Town) had pairs of tension rods at each panel point, the shorter spans (such as Monkerai) had only single tension rods at each panel point. This means that for the 100’ span, the tension rods are located in the outer two laminates of the laminated timber bottom chord whereas for the 70’ span the tension rods are in the central laminate.

This, in turn, means that for the 100’ truss, an internal steel box can be designed to miss the tension rods so that no loss of section is required in the steel at the location of the tension rods. However, this cannot be so for the 70’ truss, where the tension rod would, of necessity, penetrate through the centre of the steel box section, taking out almost the entire top and bottom flange (the central flitch is 102 mm, the diameter of tube required for the 60 mm tension rod is 76 mm).

The location of particular concern is shown in Figure 202, which is where the large diameter tension rods penetrate the bottom chord at a location of maximum bending moment. It is therefore necessary to provide external steel plates rather than an internal box (Fig 201).

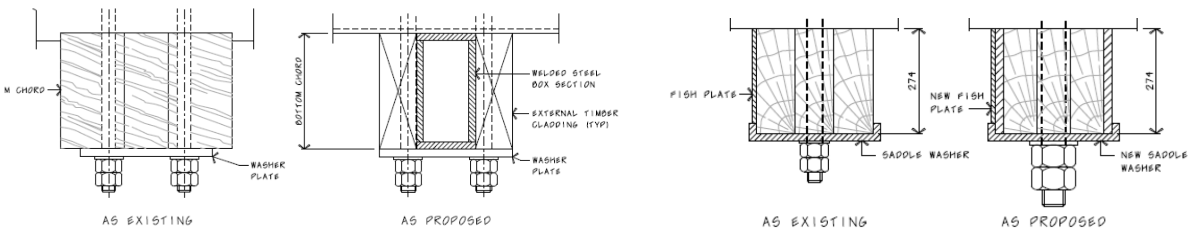


Figure 201: Comparison of tension rod layout at Clarence Town (left) and Monkerai (right)

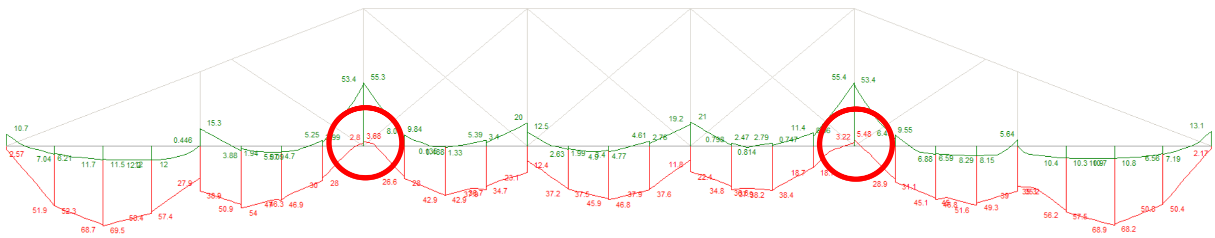


Figure 202: Design Action (Bending Moment Envelope): Ultimate Dead Load + Ultimate T44 Live Load

6.5.6 *Recommendations for Timber Members*

Therefore, changes from original details to meet current design loads are limited to the following:

- Provide external steel plates to bottom chord notched to fit steel primary cross girders
- Modify connections of counterbracing to allow for thermal and shrinkage movements
- Provide SLT deck with connections detailed to ensure lateral stiffness
- Replace timber primary cross girders with steel of hollow rectangular section
- Provide appropriately strengthened steel sway bracing at original locations only

7 Conclusion

"Engineering training deals with the exact sciences. That sort of exactness makes for truth and conscience. It might be good for the world if more men had that sort of mental start in life even if they did not pursue the profession. But he who would enter these precincts as a life work must have a test taken of his imaginative faculties, for engineering without imagination sinks to a trade. And those who would enter here must for years abandon their white collars except for Sunday.

The profession means years on the lower rungs of the ladder-shops, works, and powerhouses- white collars are not a part of the engineer uniform. On the other hand, the engineer learns through work with his own hands not only the mind of the worker but the multitude of true gentlemen among them. And men who love a fight with nature, who like to build and see their building grow, men who do not hold themselves above manual labour, men who have the moral courage to do these things soundly, some day will be above to move to town, wear white collars every day, and send out the youngsters to the lower rungs and the frontiers of industry.

It is a great profession. There is the fascination of watching a figment of the imagination emerge through the aid of science to a plan on paper. Then it moves to realization in stone or metal or energy. Then it brings jobs and homes to men. Then it elevates the standards of living and adds to the comforts of life. That is the engineer's high privilege.

The great liability of the engineer compared to men of other professions is that his works are out in the open where all can see them. His acts, step by step, are in hard substance. He cannot bury his mistakes in the grave like the doctors. He cannot argue them into thin air or blame the judge like the lawyers. He cannot, like the architects, cover his failures with trees and vines. He cannot, like the politicians, screen his shortcomings by blaming his opponents and hope that the people will forget. The engineer simply cannot deny that he did it. If his works do not work, he is damned. That is the phantasmagoria that haunts his nights and dogs his days. He comes from the job at the end of the day resolved to calculate it again. He wakes in the night in a cold sweat and puts something on paper that looks silly in the morning. All day he shivers at the thought of the bugs which will inevitably appear to jolt its smooth consummation.

On the other hand, unlike the doctor, his is not a life among the weak. Unlike the soldier, destruction is not his purpose. Unlike the lawyer, quarrels are not his daily bread. To the engineer falls the job of clothing the bare bones of science with life, comfort, and hope. No doubt as years go by people forget which engineer did it, even if they ever knew. Or some politician puts his name on it. Or they credit it to some promoter who used other people's money with which to finance it. But the engineer himself looks back at the unending stream of goodness which flows from his successes with satisfactions that few professions may know.

And the verdict of his fellow professionals is all the accolade he wants…"

- Herbert Hoover, 1954