

Menangle Underbridge Assessment
Appendix B
Fabrication and Structural Features

Fabrication and Structural Features

1. Scope

To understand how the bridge functions structurally it is necessary to consider how the bridge was built, and then how structural changes were effected. This will help understanding the reasons for much of the damage in the form of cracking appearing where it has. Further, there are unique features of the design which need to be understood in the context of any proposed inspection plan and retrofitting.

2. Fabrication

Figure B1 shows the cross-section as built in 1863, with the underslung stringers added ca. 1905-07. These stringers transferred some of the load from the cross-girders to additional ‘primary’ cross-girders as shown in Figure B2.

All of the observed cracks in the structure are located in the original cross girders and in the cleats connecting these to the bottom cell of the main girders. One hypothesis is that many of the cracks occurred during fabrication or modification of the structure, rather than through the passage of trains.

Riveting

There has been speculation on how the original spans and cross-girders were assembled. From an article in *The Engineer*, 18th December, 1863, it is clear that the bridge was shipped from Birkenhead to New South Wales as the primary elements of plates, cleats and rivets, and assembled on site. In all likelihood all the holes for connection were pattern punched in England, although the Royal Commission of 1885-86, in commenting on loose rivets and replacement refers to drilling holes¹.

The box girders appear to have been built up progressively on falsework using trestles between the main piers at ~16 m centres. In this way access into the bottom cell, a space about 450 mm square, was relatively short, to make the final connection of the cross-girders, with the cleats already attached, to the side of the bottom cell of the box girder. In this way the construction front of the main box girders was a short distance ahead of the insertion of the cross girders. (An alternative method of fixing the cleats to the bottom cell web plate before assembling the bottom cell and then attaching the cross girder appears to be physically impossible.)

The hot rivet would have to be inserted from the inside and propped to take the impact of a sledgehammer from the outside forming the head of the rivet. The process expands the rivet to fill the hole, even if the holes between plies of metal are not well aligned. (See Barker² for details of the process.) When the rivet cools it contracts from contact with the wall of the hole, and it increases the clamping force which enhances non slip shear transfer through friction.

Inspection reports frequently noted defective and poor quality rivets in the connection of the cleats to the bottom cell of the box girder. This is to be expected, as these rivets are placed *in situ*. Further, as noted by Barker, the first placed rivets tend to be the worst, the quality improving as the metal heats up and riveters establish a rhythm. Kernot³ has a catalogue of rivet defects, many of them present on Menangle Bridge.

¹ Railway Bridges Inquiry Commission, 1885-86, Report, P. 91, Special Report No. 12, Menangle Bridge

² Gary F. Barker: *A strength limit state appraisal of hot driven rivets*. Steel Construction, **34**, 1, Pp. 2-15, March 2000

³ W.C. Kernot, *On some common errors in iron bridge design*, Ford & Son, 2nd ed., Melbourne, 1906, Pp. 59-60 (See Attachment B1)

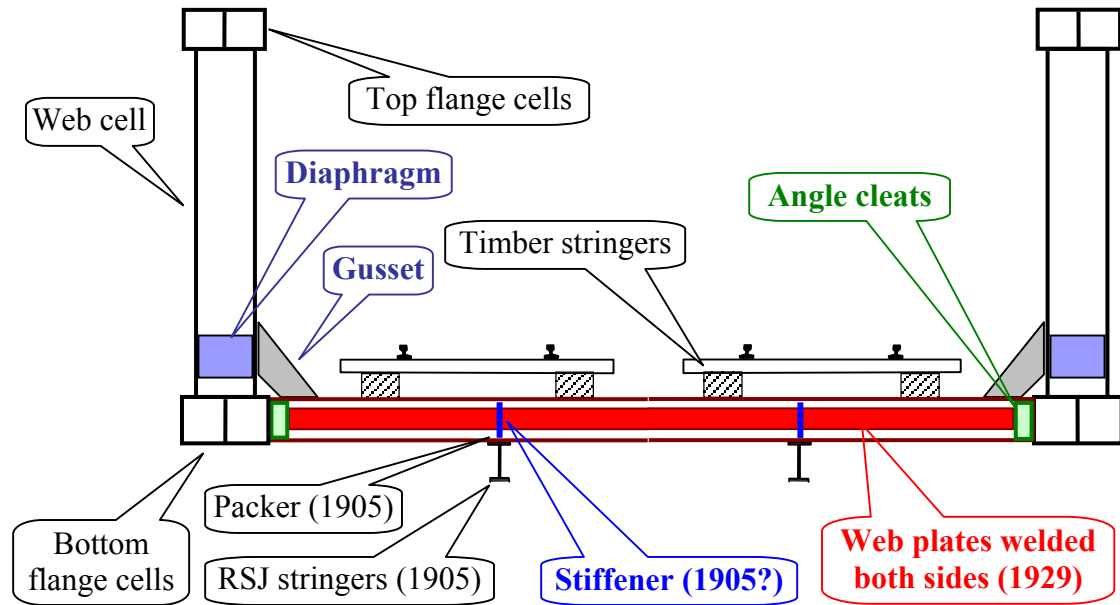


Figure B1 – Cross-section showing original cross girder with added stringer support (1905)

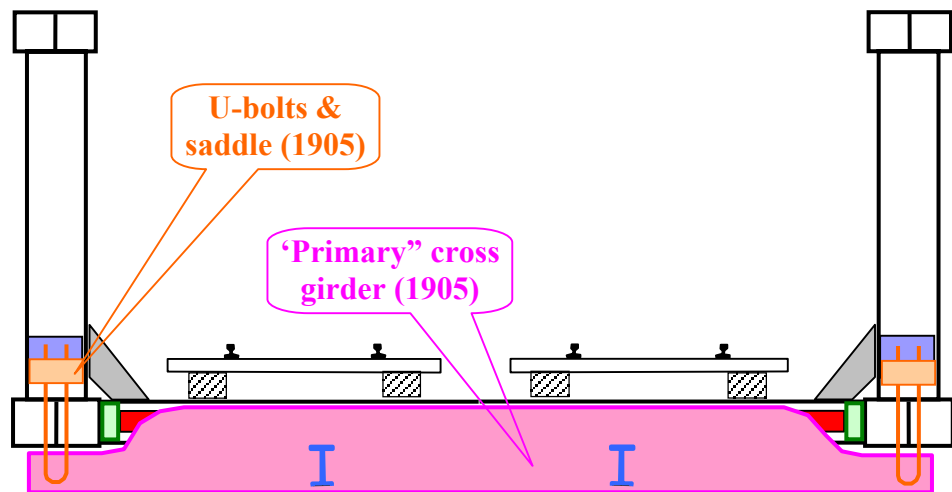


Figure B2 – Cross-section at 'primary' cross-girder, added ~1905

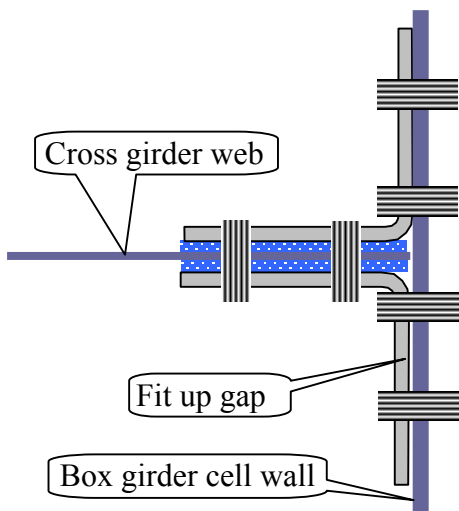


Figure B3 – Cleat attachment



Figure B4 – Crack in cleat (S2-SG27-dn-south)

An inspector's report in 1954 (see Attachment B2) notes the presence of loose rivets, poor workmanship and the inaccessibility of the bottom cells of the box girders. The report also notes the excess capacity in the cleat connections in spite of loose and missing rivets.

In addition, there is evidence of massive blows with the sledgehammer on the rivet, with the shock load transferred to the cleat. If this is sustained near the edge of the plate it can be sufficient to rupture the plate. Figure B4 shows the crack in the fillet of the cleat removed from the bridge for fatigue and material tests. The leg of the cleat on the right was connected to the bottom cell wall of the main box girder. The indentation of the rivet head in the plate to the right of the crack is evident. It is on an angle because of the difficulty of access. The crack is quite wide at the edge of the plate, about 1 mm, characteristic of a ductile tear rather than a fatigue crack.

The Royal Commission of 1885-86 noted evidence of massive blows to rivets, apparently referring to replacement rivets. However, this could not apply to the rivets in the bottom cells of the box girder, where there was not internal access. The Royal Commission also referred to gaps between plates in the assembly of the cells of the box girder, particularly on the outside. This was attributed to close fit being prevented by the sheared edges of the plates standing proud.

Fit up

The significant additional problem with this joint is the restraint offered by the part of the connection already assembled. This is illustrated in Figure B3. It is practically impossible to avoid a gap between one of the cleats and the side wall of the bottom cell. Very high flexural and tensile stresses are set up in the cleat as the rivet cools, particularly if the rivet is close to the fillet of the cleat.

Normally riveting is performed on loose pieces which have nothing to prevent the plates clamping together snug tight together. In this case this is prevented by the cleats being riveted to the web of the cross-girder first. Moreover, if there is warp or twist of the cleats a gap can be present on both sides. An additional cause of gaps lies with the punched holes, which often have a rim of metal on the punch-out side, which is rarely fully filed away, so that a gap remains. Figure B5 shows a section through a rivet hole of the cleat removed from the bridge for laboratory testing. This picture shows the lip at the edge of the hole, and the distortion of the plate. Major laminations are also evident. The hole is not punched through at right angles.

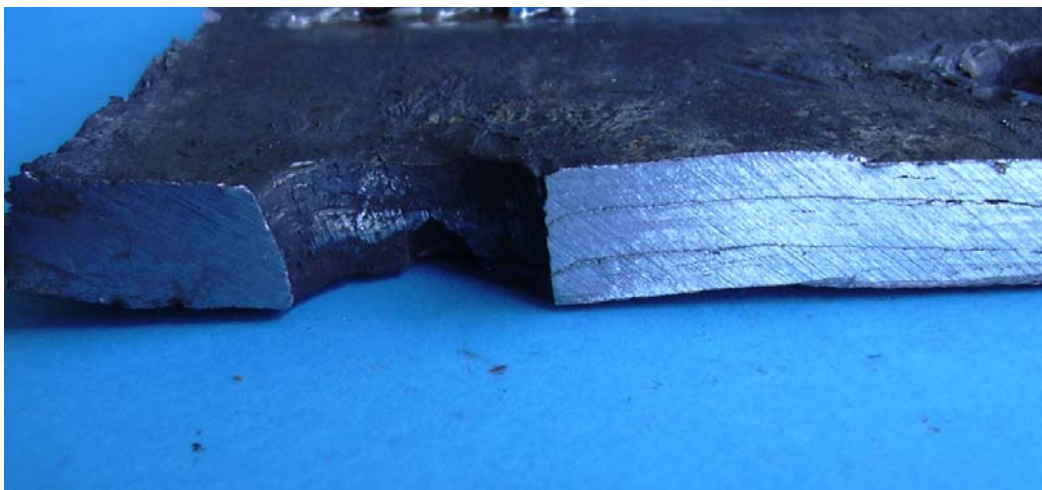


Figure B5 – Section of cleat at punched hole showing distortion and inclination of the hole

Figure B3 shows the packer plates between the cleats and the web of the cross girder. These were inserted so that the cleats could extend the full depth of the cross girder between the flanges, over the flange angles as well as the web. This thickened the web to a full inch (25.4 mm). Given the uniform spacing of the rivets along the box girder this meant that the rivets were driven into the box

girder very close to the fillet of the flange angle – so close that driving the rivets would have been very difficult. The snaps of the rivets near the fillets in the cross girder were filed away to give access to drive the last rivets of the connection into the box girder bottom cell. This is evident in Figure B6, a feature repeated many times the cleat connections.

Fortunately the hot driven rivets could be functional with poorly aligned holes, since the driving process filled the holes with metal, even if they were not properly aligned. Even with reduced heads they could retain their capacity to transfer shear by bearing action. However, reduced heads could reduce the tightness of grip, so that the rivets could work loose more easily.

Welded web plates (1929)

The original wrought iron cross girders had webs 6.3 mm (¼ inch) thick. In 1929 steel plates 9.5 mm (⅜ inch) thick were added on each side, between flange angles and stiffeners. It is not clear why these were added, although it was fashionable at the time, since the original web is adequate for shear forces from modern traffic. The butt or plug welds are rather heavy. The welding sequence stated on the drawings made the end closing welds to the vertical load bearing stiffeners the last. This sequence maximises the residual tensile stress across the plug weld to the stiffener.

3. Structural features

Antecedents

In 1840 the dominant metal for bridges was cast iron. By 1850, when the Britannia Bridge was completed, the dominant metal was wrought iron. In one step Robert Stephenson and William Fairbairn had increased the maximum span of a wrought iron bridge from 9.6 metres to 140 metres, discovered plate buckling behaviour and devised the cellular flanges which used this knowledge⁴. At Menangle the bridge was well within the knowledge and spanning capabilities of the time.

Whereas 2.19 million rivets were used in Britannia Bridge, 245,000 were used at Menangle. In each case they appear to have been hand driven, although steam driven riveting was available.

The box girders

The box girders are a robust form for redistributing stresses should some component fail. They were criticised by W.C. Kernot⁵ for their inefficiency, although this ‘inefficiency’ is one reason for their robustness. The step in width between the web cell and the flange cells results in some bending of the horizontal plates between the web and flange cells under the action of shear forces transferring to a bearing. The gusset on the ends of the cross girders appears to be an important part of the load path at bearings.

The introduction of intermediate piers by 1907 increased the load bearing capacity of these girders by more than 100%. They are also very stiff, and, being riveted, well damped. Dynamic amplification associated with free vibration will be low.

The box girders have high lateral and torsional stiffness, capable of resisting lateral forces due to wind, nosing, etc., without the assistance of deck bracing. The redundancy of elements for bending in the vertical plane also applies to bending in the horizontal plane. The girders have also demonstrated once that they act as a crash barrier, keeping a derailed train on the bridge.

The torsional rigidity is a significant factor in reducing the bending moments in the cross girders by allowing the box girder to provide end restraint to them.

⁴ Nathan Rosenberg and Waler G. Vincenti, *The Britannia Bridge*, MIT Press, 107 Pp, 1978

⁵ W.C. Kernot, *ibid* P. 20

Lateral bracing

The Royal Commission, Kernot, and various inspectors have remarked on the lack of bracing, and Kernot forecast the growth in longitudinal forces and saw the need to resist them properly. However, the consensus was that there were enough cross girders to carry the lateral force applied to the top flanges from braking and acceleration. It appears that there were no computations on the effect of longitudinal forces at first. This was a matter of concern for this investigation. The lateral forces cause bending about the minor axis and twisting of the cross girders.

Original cross girders (1863)

The original cross girders (now known as ‘secondary’ cross girders) are conventional riveted wrought iron plate girders. They are connected to the box girders by a pair of angle cleats and a gusset plate (see Figure B6). The gusset plate is an important part of the connection, both for sharing the shear transfer from the cross girder to the box girder and for transferring positive and negative moments between the two members.



Figure B6 – Secondary cross girder connection to box girder, with original and replaced gussets

When the axles of a wagon are over the secondary cross girder the middle of the span has a sagging moment and the ends have a hogging moment due to the torsional rigidity of the box girder. The ends of the cross girder rotate so that the top flange of the box girder deflects inwards. Adjacent cross girders without wheel loads directly on them have end rotations which impose a sagging moment over the whole span including the connections. Thus moment and stress reversal occurs in the whole connection. But the load sharing between adjacent cross-girders lowers the stresses in these members.

Additional cross girders (1907)

The steel cross girders added in 1907 (one for three of the original cross girders) are known as ‘primary’ cross girders. They capture some of the load applied to the secondary cross girders, which sit on packers on steel stringers framing into the primary cross girders (Figure B7).

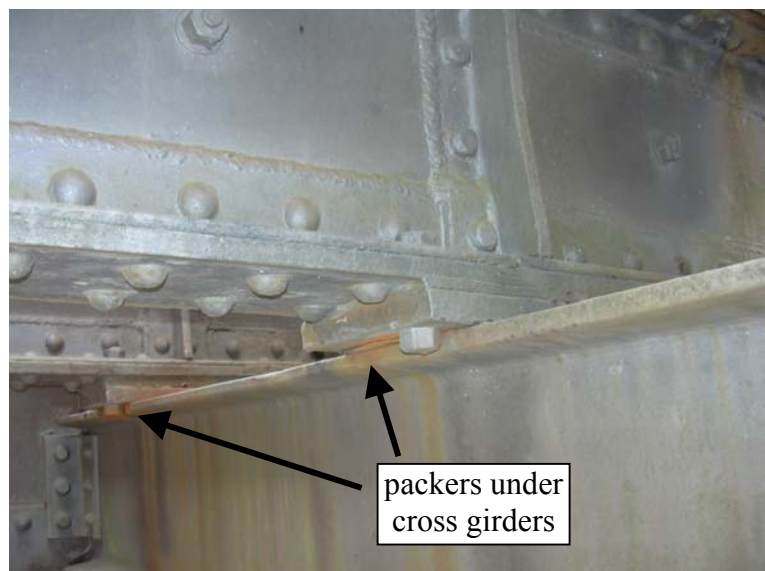


Figure B7 – Steel stringer connects to primary cross girder(left), supports secondary cross girders

The primary cross girders were supported on U-bolts, now replaced by threaded suspension rods (Figure B8). This suspension system is virtually free to sway in the longitudinal direction so the primary cross girders resist little of the longitudinal forces. Longitudinal translation of the suspension bars imposes bending moments on them.



figure B8 – U-bolt suspension and replacement for primary cross girder

Redundant load paths

This structure is rich in redundant load paths which would allow the structure to survive the complete loss of almost any component.

The main box girder is continuous over six piers, requiring three plastic hinges to form in a span (two in the end spans) for it to collapse. The loss of a complete cell flange could be survived.

The primary cross girder will remain supported after the loss of two suspension bars at a support.

The secondary cross girders can carry normal traffic should a primary cross girder completely collapse.

The end connection of the secondary cross girders includes two angle cleats and a gusset. There is ample reserve capacity if one of these is lost.

Should an end connection fail completely the secondary cross girder is supported on the steel stringers, and it will carry some load.

The primary cross girders and the adjacent secondary cross girders are adequate for normal traffic should a secondary cross girder fail completely.

There are significantly more rivets than are needed in most connections. However, some in the angle cleat connection are so close to the edge of the plate for their effectiveness to be doubted.

On some common errors in iron bridge design

W.C. Kernot

Ford & Son, second edition, 1906

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... Thus the bridges are without doubt growing weaker with effluxion of time. Meanwhile the loads they have to endure show a distinct tendency to increase. Steam rollers, traction engines, and other specially heavy loads, undreamt of at the time our earlier bridges were designed, are now common, while locomotives, with the universal call for more power, become constantly larger and heavier, and powerful continuous brakes, unknown when the earlier bridges were built, introduce longitudinal stresses of serious magnitude.

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7- *Unfavourable disposition of material for enduring bending moment.*—This is an infraction of requirement a on page 17, and is occasionally seen in the older type of box girder. A notable example is the Railway Bridge at Penrith, N.S.W., the section of which is shown in Fig. 16. Here there are four chords instead of two, and those nearer the neutral axis represent a most unfavourable disposition of material. The intermediate chords are 10ft. apart and the extreme ones 13. Consequently, first, the stress on the intermediates is only 10-13ths of that on the extremes, and as it acts at only 10-13ths of the distance from the neutral axis, the value of every square inch of metal is only $\frac{10^2}{13^2} = \frac{100}{169}$ or not much more than half of what it

would have been if placed at the extreme distance. In this way the bridge is loaded with a vast quantity of metal which performs only- about 60 per cent. of the duty it should. Further, this arrangement involves the existence of cells 18 inches square, and nearly 600ft. long, which are most objectionable from the point of view of inspection, painting and repair. Fortunately, this form of girder is now quite obsolete, so the error is not likely to be repeated.

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The older type of tubular bridges, such as the Footscray Railway Bridge, Victoria, and those at Menangle and Penrith, N.S.W., though presenting enormous surfaces to wind pressure, are invariably absolutely devoid of lateral bracing. They resist the wind simply by the resistance to bending of the main and cross girders in their weakest direction, and must be subject to extra stresses of serious amount in consequence. On the other hand, one not infrequently sees open lattice girders offering but insignificant surface to the wind, braced in the most costly and elaborate manner.

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2. Want of correspondence of rivet holes.—In the old days of punched work, this was often a serious evil. No matter how carefully the holes were marked out, the stretching of the metal in the process of punching would often cause a want of correspondence, and then violent and injurious methods were employed to force the plates into their right position, giving rise to unknown stresses of serious import. In the modern system of drilled work this evil has almost vanished. In high-class modern work the parts are carefully clamped in their intended position, drilled through all the thicknesses at once, taken apart, the sharp edges of the drilled holes scraped off by an S-shaped steel tool, then re-assembled, adjusted most carefully and accurately in their final positions, and finally riveted, preferably by some mechanical appliance giving a steady squeeze rather than a series of blows.

3. Bad riveting.—There are several varieties of this fault.

First, the heads may not be close down on the plate; second, the various plates riveted through may not be in contact, but may be actually held apart by a bulging projection on the stem of the rivet; third, the rivets may be loose to a greater or less extent; fourth, the heads may be hollow; fifth, the heads may not be symmetrically placed on the stem of the rivet; sixth, the rivets may be of lead, putty or clay. The first and second of these faults will be obvious to the eye, and the third may also be, as loose rivets often give rise to a stain of rust. It is desirable, however, to test rivets by tapping them lightly with a small hammer, placing a finger on the opposite side of the head to that tapped. Rivets should be tested at both

ends, especially if passing through many thicknesses of metal, for they may be tight on one side and loose on the other. In fact, it is very undesirable to attach more than three, or at most four, thicknesses by one set of rivets in the author's opinion, on account of this tendency

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The fourth fault occurs when the rivets supplied to the riveters have been too short. The author once met with a bridge which to mere eye inspection was perfect, but a comparatively light blow would send any rivet out, all that was left of the head struck being a thin, flat piece of iron like a small washer. The men had succeeded with gentle blows and finishing with a cup in making a head that contained less than half the metal it appeared to. Of course the rivets were all loose in their holes.

The fifth fault may sometimes be detected by looking at the edges of the plates, when the two heads of a rivet will often be seen not to correspond, as they should. The most thorough test is to cut off one head of a rivet by a chisel, and drive the rivet out by a punch. This will at once show if the heads are symmetrical and the rivets tight in their holes. A good rivet should take two or three blows of a six-pound hammer to drive it out after its head has been cut off.

The sixth fault might appear incredible, but the author has seen and tested both lead and putty rivets in some of the largest bridges in Australia, and has heard of clay rivets in a large structure in Ireland. While such things are deserving of the severest punishment to those perpetrating them, the designer is not always free from blame, for he, by needlessly placing rivets in awkward positions where they could not be properly riveted, has often led his humbler brother into temptation. The inspector of riveted work should therefore give the closest attention to any parts where there is more difficulty than usual in doing sound work.

If a serious amount of bad riveting occurs in any bridge it should be cut out little by little, at times when the structure is not heavily loaded, and replaced by sound work. As many old structures are considerably over-riveted, a few loose rivets in places where the stress is small need not cause uneasiness. ...

BRIDGE over the NEPEAN RIVER at MENANGLE.

Examined 1 and 8/9/54.

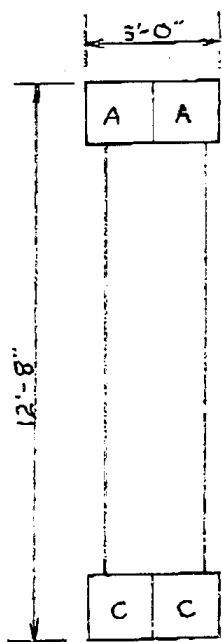
The iron spans of this bridge, which was put into service in July 1863 are the oldest in the New South Wales Railway system.

There were originally three continuous spans each of 162' resting on stone piers. In 1905 brick piers were built to support the girders at midspan and the floor system was reinforced, and in 1922 the timber approaches were replaced with steel deck spans on brick piers.

The bridge now consists of six continuous wrought iron through box girder spans each 81', fixed on pier 5 and expanding outwards from it, there are eight 25' and one 50' approach deck spans at the Sydney end and eight 50' steel deck spans at the Menangle end; there are two tracks.

The plans of the original have been lost but the available information is that piers 1 to 4 rest on rock at shallow depth, pier 5 on rock at greater depth, and 6 and 7 on piles; the piers are in good condition.

The original expansion bearings on piers 1 and 7 have each seven rollers, 4" x 42" and those on pier 3 have sixteen; the rollers are enclosed and cannot be inspected. There are oil pipes attached but their efficacy is doubtful and it may be as d that the rollers are rusty and surrounded by fine dust. The later bearings on the brick piers 2, 4, and 5 have each twelve rollers 4" x 46"; they are on pedestals well above the tops of the piers and are free of dirt but are rusty. No movement was observed under train loads but there was a slight change of length between 11.30 a.m. and 5.30 p.m.



The main wrought iron girders of box section, as in attached sketch shown diagrammatically, are spaced 28'6" centre to centre. There is little paint on the spans and the tops of the girders are rusty. The web interior can be inspected with a torch and there seems to be no corrosion except near the ends. Chord AA could be inspected with difficulty and CC not at all; there is some corrosion of the chords visible near the ends but the remainder is probably sound. It is to be noted that the plates are in places ¼" thick.

The web plates are spliced at 3' intervals where they are stiffened with tees, and there is looseness in a number of these.

There are loose rivets in the cleats connecting the cross girders to the main girders and many of the rivets have very flat snaps; however, there is a generous provision of rivets in the cleats and these faults result in no serious weakness.

The rails and transoms rest on heavy timber bearers supported on wrought iron cross girders at 3' centres. The floor system has been strengthened by the addition of deep steel cross girders at 9' suspended from the main girders, portion of the load being transferred through two lines of longitudinal steel beams.

The stresses in the floor system are indeterminate, but it and the main girders may be considered capable of safely carrying all loads as they are now doing.

There is no lateral bracing, top or bottom, but the heavy section of the box girders and the close spacing of the floor girders give the bridge ample rigidity.

The steel approach spans were designed for heavy load and have an excess of strength, the girders are rather bare of paint but the steel is generally in good condition. The lateral bracing, top and bottom consists of flat bars, which are slack and inefficient; this and the angle cross bracing are rusty and there are loose rivets, but no immediate attention is necessary.