

RECENT DEPARTMENTAL PRACTICE IN THE DESIGN
OF STEEL RAILWAY BRIDGES REQUIRED FOR
THE WATERWAYS OF THE NORTH COAST RAIL-
WAY, N.S.W., INCLUDING SOME NOTES ON THEIR
MANUFACTURE.

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The North Coast Railway Act was formally ratified just four years ago, marking an important stage in the engineering development of this State. The total length of the line will be approximately 311 miles, and for constructive purposes it has been divided into nine sections, contracts for three of which are now in progress, covering a length of 115 miles from West Maitland to Taree. Tenders are now being invited for the ninth section, covering a length of 28 miles, thus leaving 168 miles still to be dealt with.

This railway will ultimately form part of the chief line of communication with Queensland, which for coast defence and other military services will no doubt prove of great strategical value. Financially the proposition may be classed in the order of big things. The interest to engineers lies in the many practical problems that have been, and are still, being faced in its location and construction. The track will be of first-class character, capable of carrying the heaviest traffic and at the maximum speeds. Earthworks, for a considerable extent, are of a heavy character, and several tunnels are involved.

What is more cognate, however, to the purpose of this paper, is the fact that it traverses a portion of New South Wales, which is one of the best watered areas of the whole Australian continent. Tidal rivers are fairly numerous along this part of the coast, some of them being of considerable volume, and navigable in their lower reaches, while at the head of navigation in each case one finds a town of importance forming a centre of settlement for the rich lands adjoining. These rivers, with their main tributaries and their minor branches, form a veritable network of water-courses, intersecting the railway, which will finally link these towns together. There is thus a characteristic difference in the aspect of this part of the State and that of the Central and Western Divisions, where railway construction has been, hitherto, mainly proceeding, generally of the pioneer class, into which such an item as a steel bridge, of even small dimensions, is of rare occurrence.

The smaller watercourses are provided for by concrete culverts, ranging from 2ft. to 20ft. in diameter, and by timber openings from 4ft. to 24ft. span, generally in series, forming viaducts of often considerable length. Where practical conditions or economic reasons necessitate or justify the adoption of longer spans than these, steel superstructures carried on concrete piers are, and will, be provided, with approach spans of steel or timber. A list of the principal waterways is included in Table A, appended, with spans, approximate weights of steel work, and progress to date. From this it will be seen that the total amount of steelwork involved is approximately 6,100 tons, worth roughly about £190,000, in the completed superstructures. Hence the bridge question, at the outset, loomed up as an important phase in the construction of this railway and, in view of all the facts, it was decided to have a fresh set of standard designs prepared of sufficient range to cover all probable requirements. The scope of the present paper is chiefly in connection with this work, the subject of manufacture is briefly touched on, and a short review of the older bridge types in the State brings it to a close.

SINGLE VERSUS DOUBLE TRACK.

This question had necessarily to be settled at the outset, and was decided in favour of the narrower structure. The author estimated at the time that a double line bridge would cost at least 75 per cent. more than a single line, and probably this is somewhat below the mark. The former would probably be beyond traffic requirements for a great number of years, and the interest on the increased cost would gradually reduce any initial financial advantage over building two consecutive bridges, and finally eliminate it altogether. The improvements constantly being made in the manufacture of materials, in design, and in workshop practice is another good reason for prudence in the first place, that these advantages may be availed of when the fitting time arrives.

STANDARDIZING BRIDGE PLANS.

It is a matter of experience that a fairly wide range in the choice of spans for a given waterway does not, as a rule, materially affect the total cost. This being so, there is clearly no reason for unduly multiplying the variety of spans, as this would involve additional expenditure of time, labour, and money, both in the drawing office and the workshop, without any compensating advantages. The steel spans that have been adopted for the North Coast Railway range from 20ft. to 200ft., and will be found, with details of dimensions and component weights, in Table B, appended. Smaller spans are not required, as the concrete culvert and timber opening satisfactorily

meet the case, and even the 20ft. span has not often been utilised. Beyond 200ft. it becomes a question whether there is any economy in constructing single line bridges, as the wind stresses become proportionately much higher in the chords and the efficiency of the latter for carrying train loads is thereby lessened. These stresses can only be reduced by wider spacing of the trusses, which increases the weight of the cross girders, without adding anything to the carrying capacity of the bridge. This is the case until the spacing is widened to the requirements of a double line bridge. From the point of view of lateral stability, also, 200ft. span would seem to be the reasonable limit for a single line bridge, when the minimum spacing of trusses, which in this case is 17ft. centre to centre, is considered in relation to the 30ft. depth between neutral axes.

The question will probably arise, why some intermediate span between the 66ft. plate girder and the 120ft. truss span has not been provided. This gap may be termed the awkward stage in bridge spans. Above 66ft. the webs of plate girders become unwieldy things to handle, both in transshipment and in shop processes, and are liable to develop strains and irregularities which are difficult to eliminate.

The manufactured sections again would be likely to give a lot of trouble in forwarding to the bridge site, as this often has to be done per medium of rough temporary roads. If the number of sections are increased to reduce individual weights, then the amount of field riveting is increased, which is not a desirable feature. In through girders the difficulty of properly bracing the top chord against flexure becomes accentuated as the depth of the girder increases, while deck girders become less easy of adoption, on account of reduced clearance above flood levels. There are, on the other hand, some good reasons for not reducing the truss span below 120 feet, if of the through type, the chief of these being the restriction of efficient portal, and sway bracing, unless the truss is of abnormal depth. Deck truss spans of 80 feet and 100 feet length would be probably the most satisfactory type to adopt if the levels permitted, but so far the occasion for such has not yet arisen.

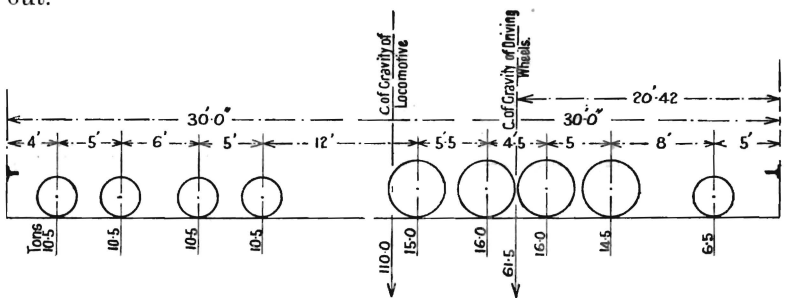
DEAD LOADS.

The permanent way details are practically the same in all cases. Rails are of the flanged type, 40 feet long, 80lbs. per lineal yard, jointed with standard angle fish-plates weighing 28lbs each, and 7-8in. diameter fish-bolts, and spiked to transoms with 7-8in. diameter stock spikes. Transoms, invariably of ironbark, are 9ft. 6in. long, by 10in. wide; the thickness varies from 6in. to 7in., according to the camber, which is to some extent reduced in the laying of the permanent way; it is still further increased on curves to give the required super-

elevation of the outer rail, tapered transoms being used. In this case, an additional rail is laid, as a guard, outside the outer rail, with a 6in. space between the heads. Transoms are held down to the girders supporting them by 7-8in. diameter bolts, which pass through the flanges and lie alternately on each side of the rail, the spacing being about 18in. c. to c. Excluding transom bolts, but including an extra pair of rails and fastenings to act as guards, a possible future addition only, as guard rails are not, in this State, usually laid on straight bridges, the total average weight of above will be approximately 6 tons per 40ft. length, or .15 tons per lineal foot of track. The detailed weights of the structure itself for the various spans are given in Table B, appended.

LIVE LOAD.

The heaviest locomotive in use in this State at the time designs were being prepared was that known as the Australian Consolidation Goods, or T class, weighing $107\frac{1}{4}$ tons in full steam; of which $59\frac{1}{2}$ tons were on the four driving axles, $15\frac{1}{2}$ tons being the single maximum axle load. It was ascertained that the Railway Commissioners had no intention of introducing heavier types than this, and it was further considered that the practice of duplicating the engine power on difficult sections would cover traffic requirements for a long way into the future. To simplify calculations, the wheel loads and spacings were somewhat rounded off, bringing the total weight up to 110 tons, and the total length, buffer to buffer, to 60ft., while the maximum axle load was made 16 tons. This modified type, shown hereunder, was adopted as the basis of calculation throughout.



— **DIAGRAM OF 110 TON LOCOMOTIVE** —

The T Class still remains the heaviest class of goods engine in service, but during last year some heavy passenger locomotives were introduced, chiefly for the purpose of hauling the Melbourne Express over the stiff portions of the Southern Line. These are known as the N Class, and weigh 108 tons in full

steam, the maximum axle load being 16.65 tons, and the total weight on the three driving axles 50 tons, on a wheel base of 14ft. As freight trains in this State rarely travel faster than 20 miles per hour, which speed may be more than doubled by the passenger service, it is probable that these N Class engines will produce much higher stresses in bridge structures than the T Class, but not higher than the adopted standard type after due allowance for impact, a matter which will now be considered.

IMPACT.

This subject has been very fully investigated in a paper* contributed to this Society about nine years ago by Professor Warren and Mr. Dare, and only brief reference is necessary here. The formula adopted therein, which was that included in the Sydney Harbour Bridge specification, is based on the assumption that the maximum percentage of live load to be added as impact need not exceed 75, and that its value generally should vary in accordance with the ratio of the live load stress to the sum of the live and dead load stresses. Expressed in symbols the total impact stress would be, in any member, $I = L \times .75 \times L \div L + D$. This formula has been adopted in the calculations for the bridge designs under consideration, and the author sees no valid reason for departing from it. The formula adopted by the American Railway Engineering and Maintenance of Way Association in 1906, which has now become standard practice in the United States is: $I = 300 \div (S + 300)$.

Where S = length of live load on track to produce max-stress in member under consideration.

This gives somewhat higher values than the preceding formula, as the following table shows. Average values given—

MEMBER.		$I = 75 \% \frac{L}{L + D}$	$I = 100 \% \frac{300}{S + 300}$
32 ft.	Span—Chords	67 per cent.	90 per cent.
66 „	„ „	58 „ „	82 „ „
120 „	„ „	54 „ „	71 „ „
120 „	„ Web	60 „ „	78 „ „
157 ft. 6 in.	„ Chords	52 „ „	66 „ „
157 „ 6 „	„ Web	59 „ „	74 „ „
200 „ 0 „	„ Chords	48 „ „	60 „ „
200 „	„ Web	56 „ „	68 „ „

* "The Determination of Working Stresses for Bridges," by Prof. W. H. Warren, M. Inst. C.E., M. Am. Soc. C.E., and H. H. Dare, M.E., Assoc. M. Inst. C.E. Oct. 18th, 1901.

It might be said roughly that the departmental formula takes into account the element of mass, and the American formula the element of time. The riveted type of truss, especially with the rigid sections adopted for tension members, leads to much heavier structures, which are at the same time less susceptible to vibration than the pin-connected, eye-bar type common in America. Much lower speeds, too, are the rule in this State, so that it would seem reasonable to expect some such difference as above in the impact allowances. The chief difference, in fact, between the two methods arises from the assumed possible maximum of 75 per cent. in the one case and 100 per cent. in the other, the former being based on Prof. Turneaure's experiments. With regard to the question of speed, the author has noted in a number of departmental tests that the increase in deflection has been very small, often nil, for an engine speed of about 20 miles per hour. Prof. Turneaure, as the result of his experiments, arrived at a conclusion which practically confirms this. Hence, provided this speed is not exceeded, there seems no reason why locomotives weighing from 20 to 30 per cent. more than the present T Class should not be safely run across these bridges, as the actual stresses would probably not exceed those allowed for.

WIND LOADS.

The wind pressure on the structure has been assumed at 50lbs. per square foot on unloaded spans, and 30lbs. per square foot when loaded. The surface exposed is considered to be twice the area of the truss, and one and a-half times the area of the floor as seen in elevation.

On the train the pressure is treated as a rolling load of 300lbs. per lineal foot, the area of the train being assumed at 10 square feet per foot. The centre of pressure is assumed to act 5ft. from rail level, and the overturning moment produces an additional loading of 300lbs. per lineal foot on the leeward stringers and deck girders, since the rails are also approximately 5ft. centres. The additional loading on the leeward trusses and through girders varies according to their spacing centre to centre.

UNIT WORKING STRESSES.

The following were adopted, the values being in tons per square inch:—

Tension	7.0
Compression	7.0 — $.03 l \div r$
l—length between neutral axes; r—radius of gyration.	
Shear—Shop work	5½
Shear—Field work	4½
Bearing—Shop work	11

Bearing—Field work	9
Bearing—Expansion Rollers, 300lbs. per lineal inch \times diameter in inches.	
Bearing—Concrete, 300lbs per sq. inch.	

The above apply to any combination of dead, live and impact stresses, or to stresses due to wind alone. For combinations of all four items, the above stresses to be increased 20 per cent. In the solid webs of plate girders, unit stress not to exceed that given by formula.

$$\frac{12,000}{1 + \frac{1}{3000} \left(\frac{d}{t}\right)^2}$$

Where "d" = clear distance between chord flanges or stiffeners; and "t" = thickness of web.

This determines the spacing of the stiffeners.

The above values agree very closely with those adopted by the American Railway Engineering and Maintenance of Way Association in 1906, with the exception of the final formula for web plates, which has been replaced by the following:—

$$d = \frac{t}{40} (12,000 - S)$$

Where S = shear on the section per sq. inch in lbs.

This gives much wider spacing of the stiffeners than the formula adopted.

METHOD OF COMPUTATION.

Dead loads have been treated as uniformly distributed or concentrated at panel points as the case required. In truss spans the weight of the floor, lower wind-bracing and half the truss has been considered as acting at the bottom panel points, the balance of the weight at the top panel points.

The live load has been treated throughout on the concentrated wheel load method. In the opinion of the author, this method is preferable, for railway bridge designing, to that of the equivalent uniformly distributed load, or any variation of the same, on the grounds of closer adherence to actual working conditions, general application to all cases and consistent results. The subjoined table shows a series of comparisons between the two methods, the uniform load being calculated on the basis of the central bending moment obtained by the use of the wheel load method. The end and central shears have been determined on a rigid basis, the maximum result being obtained when the load extends over the panel in question to the same amount that it extends over the whole span, the proportion being $m \div n - 1$. Where m = number of loaded panels and n = total number of panels. All values are in tons.

SPAN.	MEMBER.	LIVE LOAD.	STRESSES.	Equiv. Uniform Distrib. Load. per lin. ft.	PERCENTAGE. Increase of Stress.
		Concentrat'd Load Method.	Uniform Load Method.		
32ft. 0in.	Chord at Centre.	+ 43·4	+ 43·4	1·38	Equal
	Web at End	24·0	21·5		11·6
66ft. 0in.	Chord at Centre	+ 77·6	+ 77·6	1·06	Equal
	Web at End	+ 37·8	34·0		11·2
120ft. 0in.	Chord at Centre	+ 69·3	+ 69·3	·92	Equal
	Batter Brace	+ 64·0	+ 60·1		6·5
	Diagonal at Centre	- 24·4	- 21·6		12·9
	" " "	+ 12·2	+ 9·6		27·1
157ft. 6in.	Chord at Centre	+102·5	+102·5	·89	Equal
	Batter Brace	+ 82·9	+ 78·4		5·7
	Diagonal at Centre	- 22·7	- 15·1		50·0
200ft. 0in.	Chord at Centre	+143·3	+143·3	·86	Equal
	Batter Brace	+102·2	+ 98·0		4·3
	Diagonal at Centre	- 37·4	- 32·0		17·9
	" " "	+ 21·6	+ 13·8		56·5

It is evident from the above that the uniform loads required to produce equivalent end shears would be greater than those assumed, and for centre shears very much greater still.

The discrepancies can be reduced by calculating the uniform load on the bending moment at the quarter point, using a separate loading for shears, and considering, for a maximum, all panel points to one side fully loaded, and to the other unloaded. A better modification is to consider an excess loading at the head of the train representing the driving wheel base. It must be noted, however, that any variation from the method in its simplest form reduces the one advantage it possesses, the saving of time and labour, which makes it servicable for preliminary calculations. By the use of load and moment tables and diagrams, such as that appended, the calculations on the concentrated load method are so greatly facilitated that no practical objection on this score can be urged. The use of the diagram is twofold:—(1) To find the position of wheel loads for maximum effect at any point; (2) to show the value of such effect considered as a moment or a shear. Its application depends on certain well-known rules, which are as follows:—

1. For maximum bending moment at a panel point the average unit load on the shorter segment must equal the average unit load on the whole span.

2. For maximum shear in a panel the load on same must equal the average panel load for the whole span, the panels to one side or other of that in question being unloaded. The two values thus obtained will represent the positive and negative maximum respectively.

