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The Design of Small Railway Underbridges with Special Reference to Erection and Maintenance under Traffic

Die Berechnung kleiner Eisenbahnbrücken für Unterführungen unter besonderer Berücksichtigung von Montage und Unterhalt ohne Betriebsunterbruch

Le calcul des petits ponts de chemin de fer en passage inférieur, du point de vue particulier du montage et de l'entretien sans interruption du trafic

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Introduction

This paper deals with modern practice in the design of underline girder bridges on British Railways with particular reference to the procedure adopted on the Western Region with which the Author is directly concerned. The choice of design and the form of construction adopted is influenced by the method of erection, which in reconstruction work is governed by the type of structure being replaced, the density of traffic and the topographic features of the site.

Design Loading

Complying in general with the requirements of B.S. 153—1937, bridges are designed to carry loadings A, B and C given in the Bridge Stress Committee Report of 1928[1]). These loadings are equivalent to the maximum effect of a combination of British Standard Unit Loadings (see British Standard 153: Part 3: 1937) with hammer blow effects as follows:

Loading A: 20 B.S. units with 5.0 tons hammer blow at 5 revolutions per second.

Loading B: 16 B.S. units with 12.5 tons hammer blow at 5 r.p.s.

Loading C: 15 B.S. units with 15.0 tons hammer blow at 5 r.p.s.

Normally bridges are designed on the assumption that a speed equivalent to or greater than 6 revolutions per second will be attained; but, where the situation is such that the maximum speed will never reach such limits, the design loading is reduced in accordance with the maximum attained at 4.5, 3.0 or 1.5 r.p.s. as the case may be.

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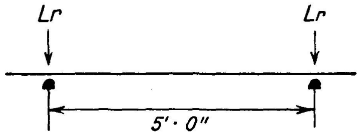
[1]) Department of Scientific and Industrial Research: Report of the Bridge Stress Committee, London 1928.

These loadings include impact effects due to hammer blow and rail joint. Allowance for lurching, applicable to girders supporting one rail only and to other longitudinal girders according to the positioning of the track between them, is made in accordance with fig. 1.

Lr = allowance per rail. The value of which at various speeds is as follows:

6 r.p.s.	25%	of static load on one rail	Note: The static load is 20 units of B.S. loading.
4·5 ,,	$18^3/_4$%	,, ,, ,, ,, ,, ,,	
3·0 ,,	$12^1/_2$%	,, ,, ,, ,, ,, ,,	
1·5 ,,	$6^1/_4$%	,, ,, ,, ,, ,, ,,	



Allowance for main girders.

Single track

$$Lm = Lr \times \frac{5 \cdot 0}{S}$$

Double track

Allowance for lurching to be made for one track only but applied to each main girder

$$Lm \doteq \frac{Lr \times 5 \cdot 0}{S} \text{ as per single track}$$

No allowance for lurching to be made for centre girder of 3-girder bridges or for cross girders.

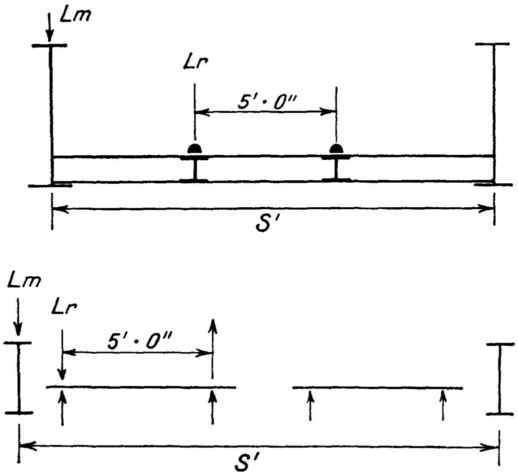


Fig. 1. Lurching method of calculation

Longitudinal Distribution

Allowance is made for the longitudinal distribution of the loading through the track on spans less than 10 feet in length. The condition giving maximum loading under loading B, two wheels 6 feet apart with hammer blow at 6 revolutions per second is shown in fig. 2.

The point load at B is distributed along the rail, over the width of the sleeper, and through the ballast. The rail is considered to be a beam supported on a continuous elastic foundation, the distance between sleepers being small compared with the wavelength of the deflection curve of the rail itself[2]). Under "very firm" conditions such as obtain on bridge decks, the load under the rail required to depress a sleeper one inch is estimated to be 240 tons[3]). Applying this value to the theory of continuous elastic foundation, the distribution of loading on the railbearer is as given in fig. 3.

[2]) "Strength of Materials", Part II, Chapter 1, by TIMOSHENKO.

[3]) "Report on evaluation of bending moments occurring in rails under traffic conditions", British Railways Research Department, April 1948.

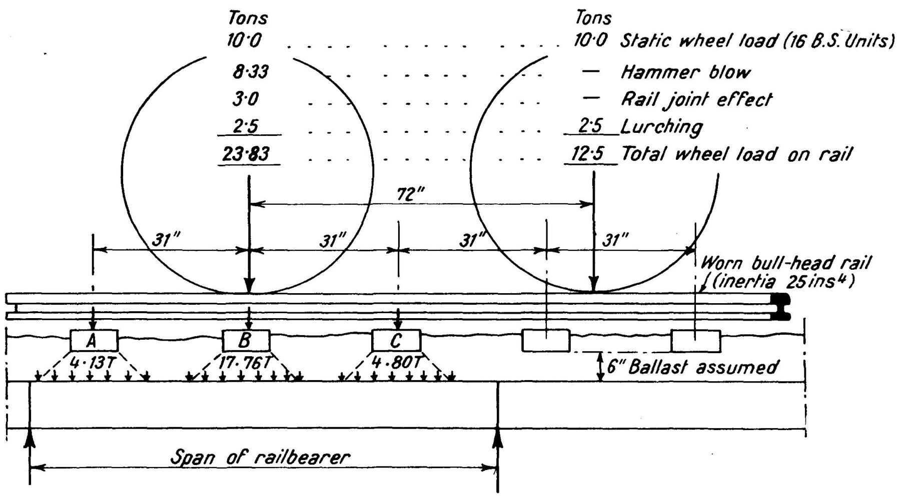


Fig. 2

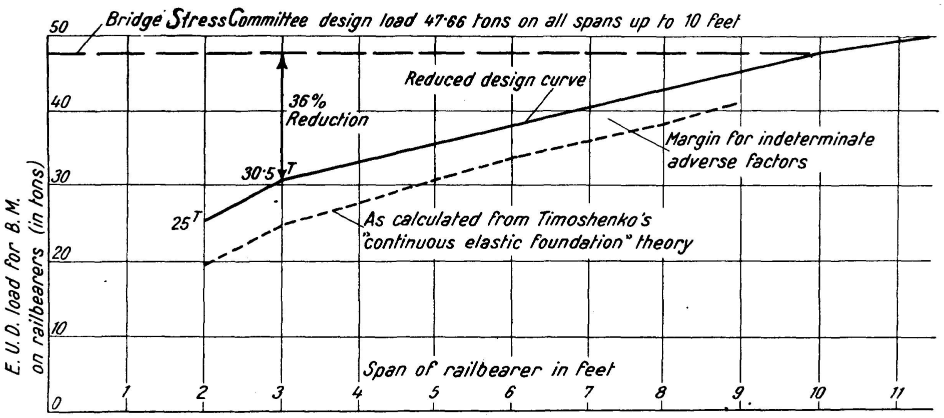


Fig. 3

Ballasted Tracks and Deck Waterproofing

So far as the construction depth available permits, provision is made for the normal cross-sleeper ballasted tracks to continue across bridges. This simplifies track maintenance and gives better running, especially at the approaches, than is the case where a change is made to longitudinal timbers or to non-ballasted tracks.

The minimum depth of ballast under the sleepers is generally 6 inches, but in exceptional cases this may be reduced to 4 inches and then a special fine ballast is used.

The provision of ballasted tracks on steel or reinforced concrete bridge decks necessitates the laying of an impervious medium protected against injury, from the stones of the ballast and plate-layers' tools, by a layer of third quality brindled tiles. This waterproofing necessarily increases the construction depth by some 2 or 3 inches, and total occupation of the line is required while it is being laid. The waterproofing is shown in fig. 4. It can be laid only in the dry so that during rainy weather the work must be protected by tarpaulin "tents" stretched above the workmen.

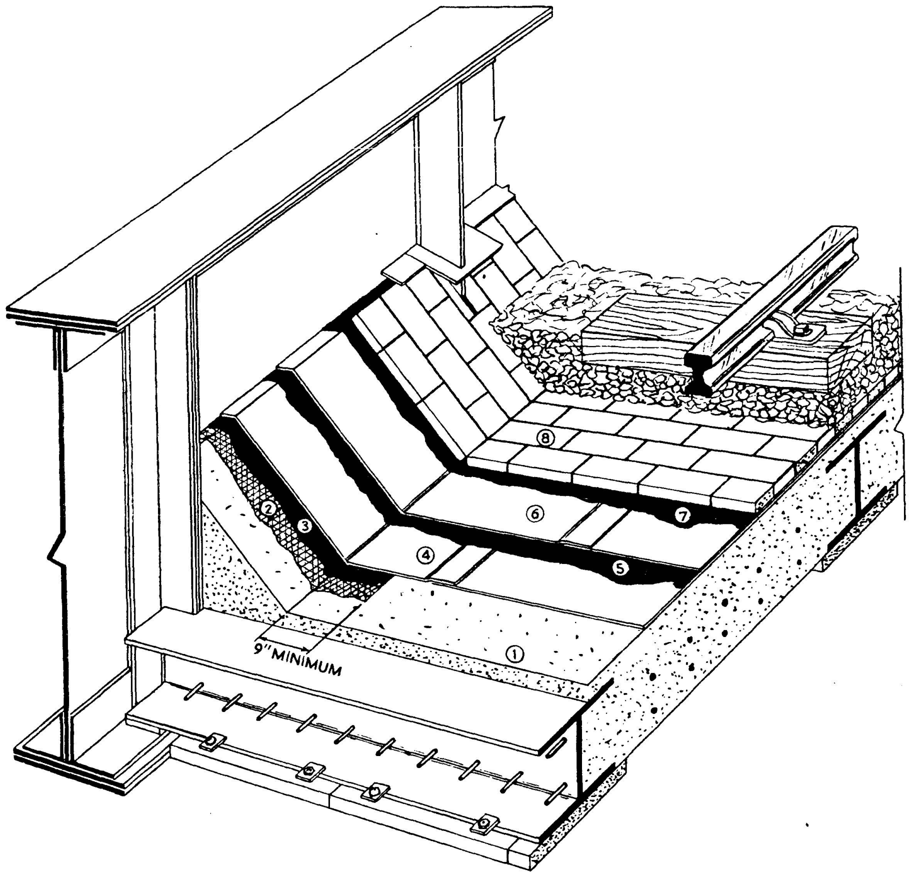


Fig. 4. Standard waterproofing

(1) Concrete to be 1·2·4 mix. $^3/_4''$ aggregate, and to comply with B.S. Code of Practice C.P. 114. (1948). (See note (b) below).
(2) Bitumen primer applied cold with a brush (see note (a) below).
(3) Hot bitumen compound as 7. above (see note (a) below).
(4) Bitumen sheeting, woven hessian base. As for (6) above. 4" lap seared at joints.
(5) Hot bitumen compound as 7. above.
(6) Bitumen sheeting, woven hessian base. e.g. Pluvex n° 1 or other approved. 4" lap seared at joints.
(7) Hot bitumen compound, melting point between 180° and 200° F. applied at an easy pouring consistency and not allowed to boil.
(8) Third quality brindle tiles $9'' \times 4^1/_2'' \times 1^1/_4''$ laid smooth face downwards. Joints run in with hot bitumen.

Note (a) The first layer of bitumen sheeting to be laid loose except at edges and haunches, where it will be primed and stuck in hot bitumen to a width of not less than 9 inches.
(b) On all-steel decking, (C) grade concrete, $1^1/_2$ inches thick, laid to falls.

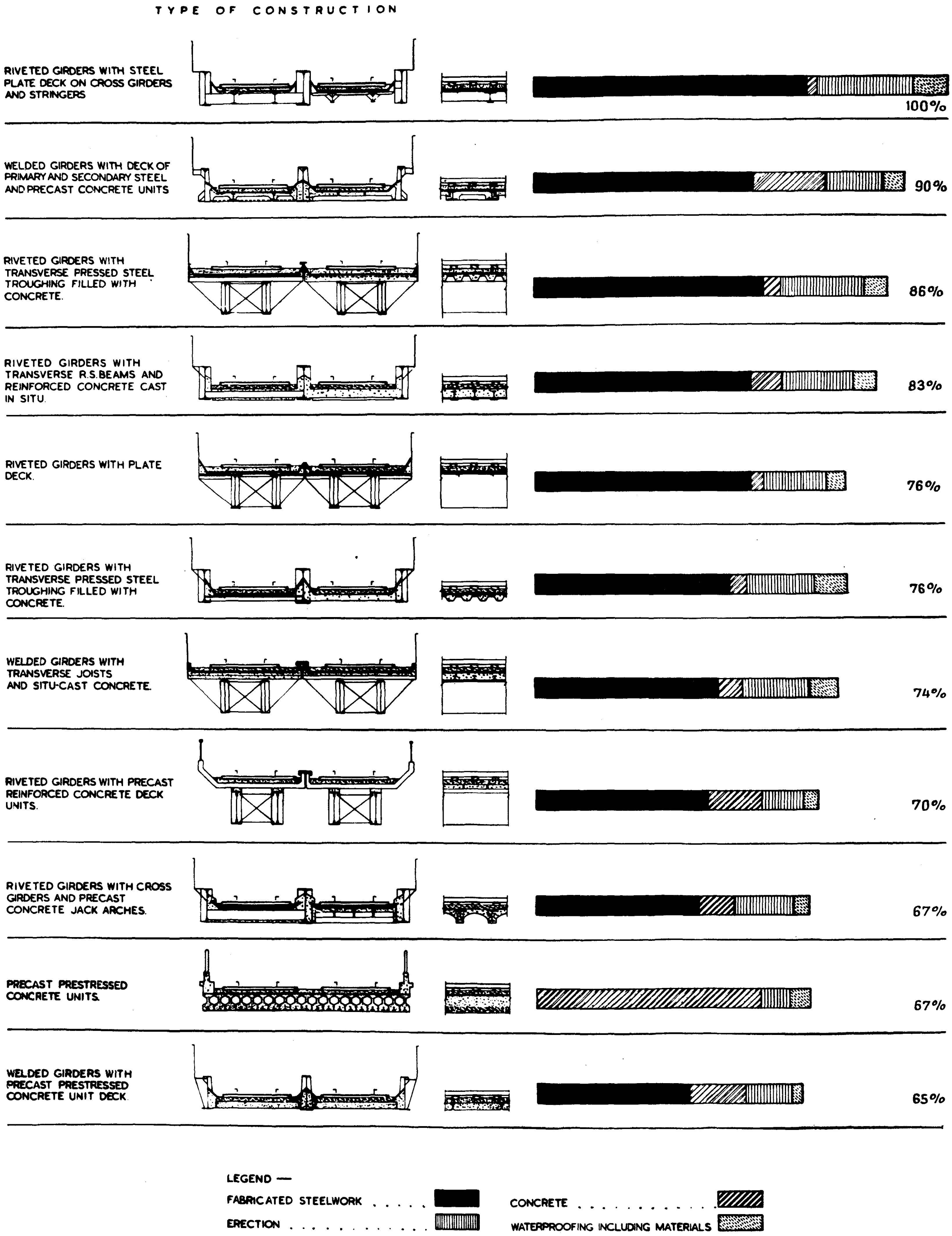


Fig. 5. Comparative costs of superstructures of double track 40[ft] square spans (November 1951)

The Choice of Type of Construction

A comparison of the costs of various types of construction for a double track 40 ft. span is given in fig. 5. Costs are based on prices ruling in November 1951.

Deck-Type Spans

The forms of construction used for short deck-type spans up to certain limits are as follows:

a) Longitudinal pressed steel troughing up to 20 feet.
b) Units consisting of rolled steel beams and precast concrete up to 30 feet.
c) Precast reinforced concrete slab units up to 15 feet.
d) Multiple precast reinforced concrete U-beams up to 45 feet.
e) Longitudinal built-up rail-bearing steel trough girders where the construction depth is very restricted, up to 30 feet.
f) Multiple precast prestressed concrete beams with transverse post-tensioning, up to 50 feet.

Precast Prestressed Concrete Beams

The precast prestressed concrete beams are prestressed under the Freyssinet or Lee-McCall systems. The beams are made in one of the railway depots. The specified crushing strength of the concrete is 7,500 lb. per square inch at 28 days and 4,800 lb. per square inch at 7 days. The design working stress in the precast concrete is 2,500 lbs. per square inch and the stresses in the cables and bars are:

	Ultimate load in tons per sq. inch	Working load in tons per sq. inch
Freyssinet cables	100—110	60—62
Lee-McCall bars	64—72	35

The longitudinal multiple-beam precast prestressed concrete span carrying a cross-sleeper ballasted track requires a construction depth equal to only $\frac{1}{20}$ of the span. The relatively shallow construction, coupled with the advantages of the small quantity of steel involved, the greater expected life and the small maintenance charges likely in the future, make this form of construction attractive. Apart from the caulking of joints between beams, prestressed concrete decks are not waterproofed.

Deck-Type Plate Girder Spans

Longer deck-type spans consist of mild steel (or, for spans exceeding 80 feet, high tensile steel) girders with a decking of:

a) steel plates either buckled or flat,
b) transverse steel troughing,
c) transverse rolled steel joists with situ concrete, or
d) precast reinforced concrete "well-deck" units.

Where conditions permit, the superstructures for each track are kept separate to avoid transverse distortion due to the unequal loading of adjacent tracks. In steel girder spans up to about 65 feet long, the two main girders together with the sway and lateral bracings can be completely fabricated as a

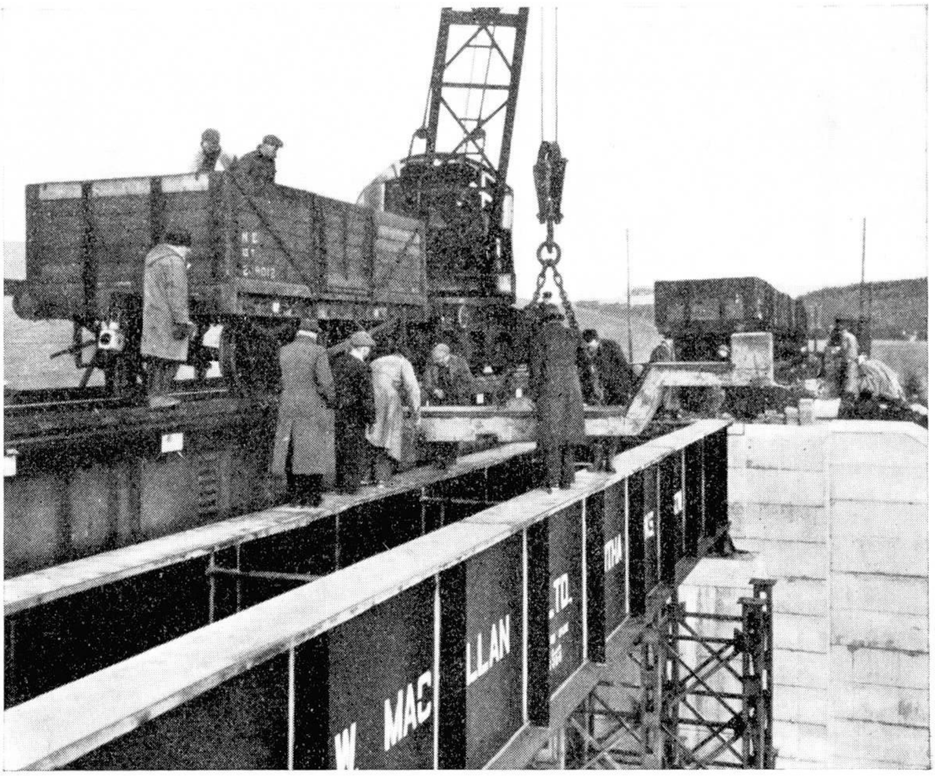


Fig. 6. Placing precast well-deck units on welded plate girders

unit before despatch from the shops, leaving only the attachment of the decking to be done at site. The methods of securing the "well-deck" units to the main girders vary a good deal. In Scotland where 6 new spans (fig. 6) carrying a main line over waterways were built in 1949, lugs are incorporated on the underside of the units to locate them on the flanges of the girders. The girders are arc welded and the flanges offer an uninterrupted level surface on which the units rest. Three layers of bituminous sheeting are interposed between the concrete units and the steel girders. Each unit is held down with hook bolts which grip the flanges of the girders.

On the Western Region some spans under construction have riveted main girders. Here a layer of concrete $7\frac{1}{2}$ inches thick is cast on the flanges of the girders before erection. The well-deck units are located by bolts passing

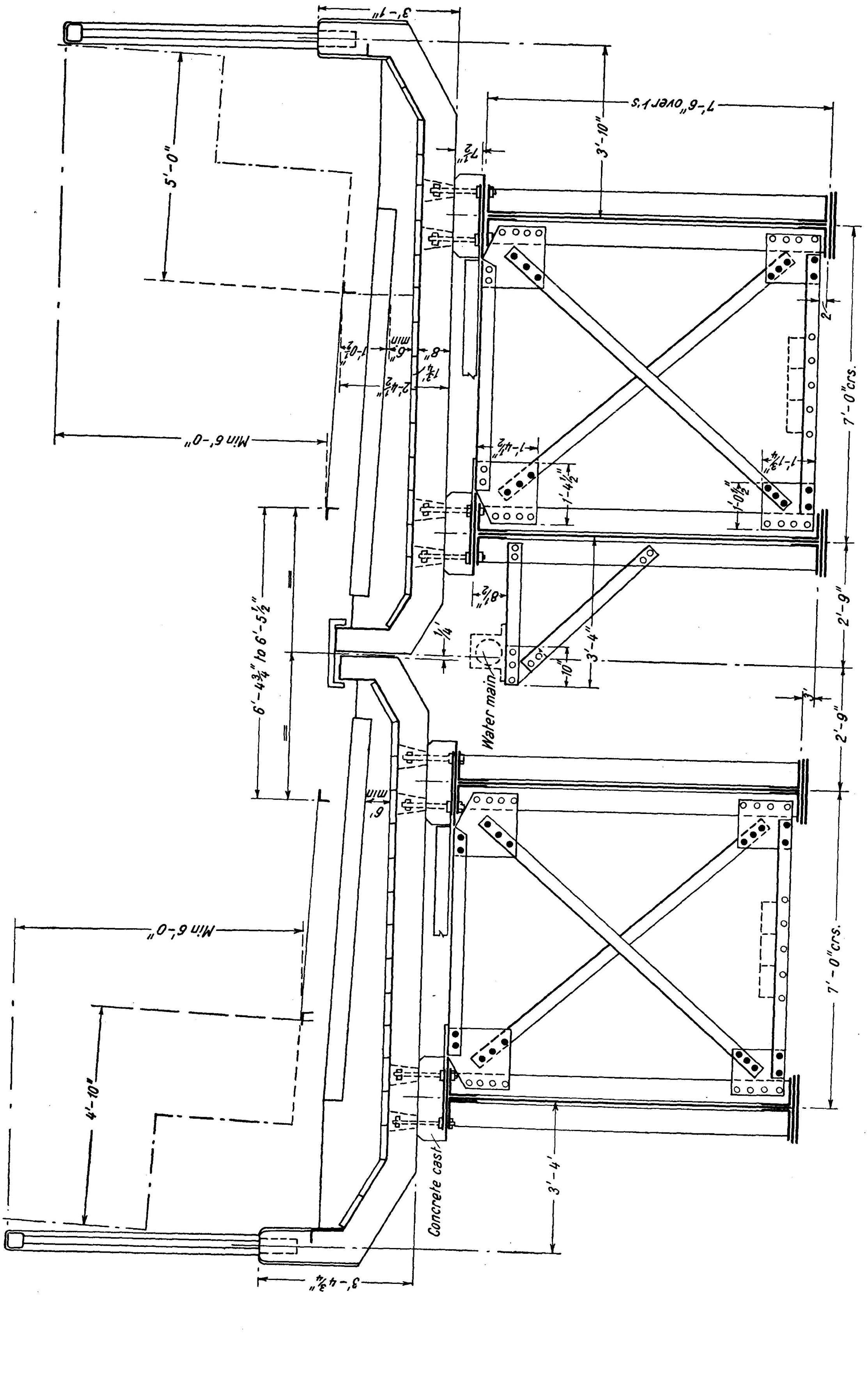

Water main
Concrete cast
7'-0" crs.
7'-0" crs.
2'-9"
2'-9"
3'-4"
3'-4"
3'-10"
5'-0"
4'-10"
Min 6'-0"
Min 6'-0"
6'-4¾" to 6'-5½"

Fig. 7

through the flanges and the layer of concrete. The layer of concrete is broken by paper joints at intervals of 6 feet and does not form part of the compression flanges of the main girders. The arrangement is shown in fig. 7.

When the decking forms a continuous part of the structure as a whole (as in the case of transverse joists and situ concrete) due provision is made for the transfer of shear stresses between the main girders and the decking. This is shown in fig. 8.

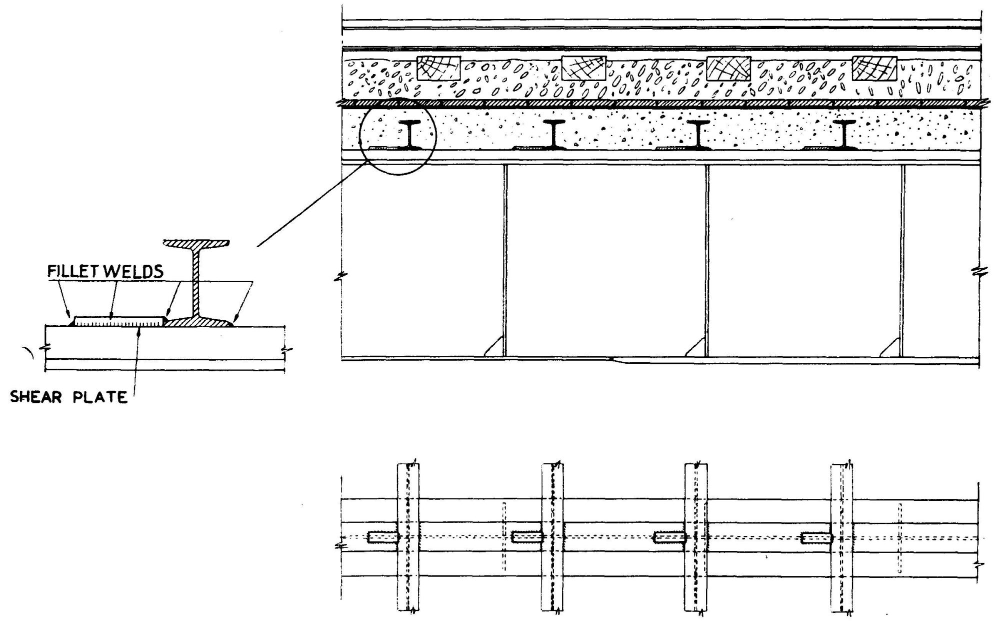


Fig. 8

Continuous decking is not counted as a part of the main girders and no relief in flange stresses is afforded in the calculations apart from the obvious reduction in the ratio of length to breadth of compression flanges. The precast "well-deck" reinforced concrete deck units are in widths of about 3 feet and, with the transverse joints between run in with bituminous sealing composition, the deck is not regarded as continuous.

Through-Type Spans

Because the construction depth available is often very restricted, the through-type plate girder bridge is perhaps the commonest form of construction on British Railways. The decks of through-type spans generally consist of one of the following:

I. cross girders, stringers and steel plate floor;
II. transverse pressed steel troughing;
III. transverse steel joists with situ-cast concrete;
IV. a combination of (a) primary units consisting of steel cross girders and stringers encased in concrete, precast before assembly, and (b) secondary units of precast reinforced concrete;

V. steel cross girders and precast reinforced concrete jack arches;
VI. prestressed concrete units.

Although in the double track bridge it is desirable to have separate superstructures for each track, space is seldom sufficient and such spans generally have a centre main girder common to both tracks. In this, the 3-girder span, special attention is paid to the connections of the decking which are now designed to develop the full end-fixity of the deck units. Sufficient attention has not always been given to this feature. In the past cross girders have been designed as simply-supported beams, although in actual fact some degree of end-fixity has generally been provided. Because of weakness, these connections have often worked loose and maintenance troubles have ensued.

Steel Plate Decking

Steel plate decking carried on cross girders and stringers, for many years the standard form of construction, is no longer in favour because of the costly painting work involved in the maintenance of the underside. Moreover, site assembly involves a good deal of field riveting; and if this is to be carried out under traffic following piecemeal assembly, the field work becomes very protracted. A layer of asphalt or a carpet of concrete must be laid all over the steel plate decking, so as to cover the rivet heads and provide a suitable surface to receive the waterproofing.

Transverse Steel Troughing

Transverse steel trough decking, widely used in the past, has not been without maintenance difficulties. The troughing, generally resting on the bottom flanges of the main girders, has been cleated to the web at the top of the troughing only, as shown in fig. 9. No doubt this arrangement has led to some flexibility at the joint, and this, while very desirable in one way, has led to movement of the bottom of the troughs on the flange of the girder. This movement has quickly led to corrosion, the troughs have gradually dropped and the cleat connections at the tops of the troughs have become overstrained. A typical example of this trouble is shown in fig. 10.

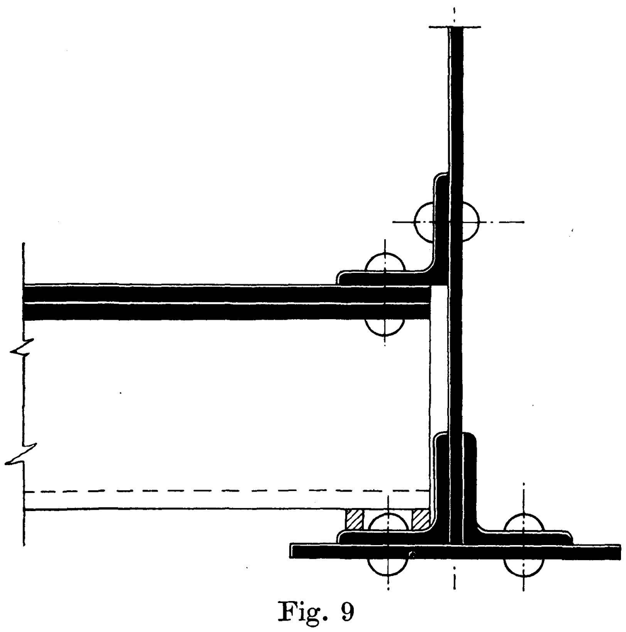


Fig. 9

In addition to the difficulty of providing a satisfactory connection between the troughing and the main girders, there is the further difficulty, when waterproofing is used, of maintaining a satisfactory watertight joint against the girder.

Transverse trough decking certainly provides one of the shallowest forms of construction and for this reason it is still used when the construction depth available is very limited. In a recent example (figs. 11 and 12) the troughing

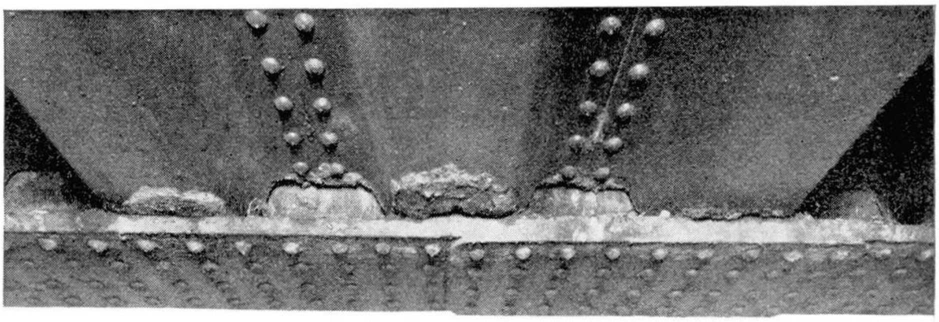


Fig. 10. The failure of the connection between transverse pressed steel troughing and a main girder

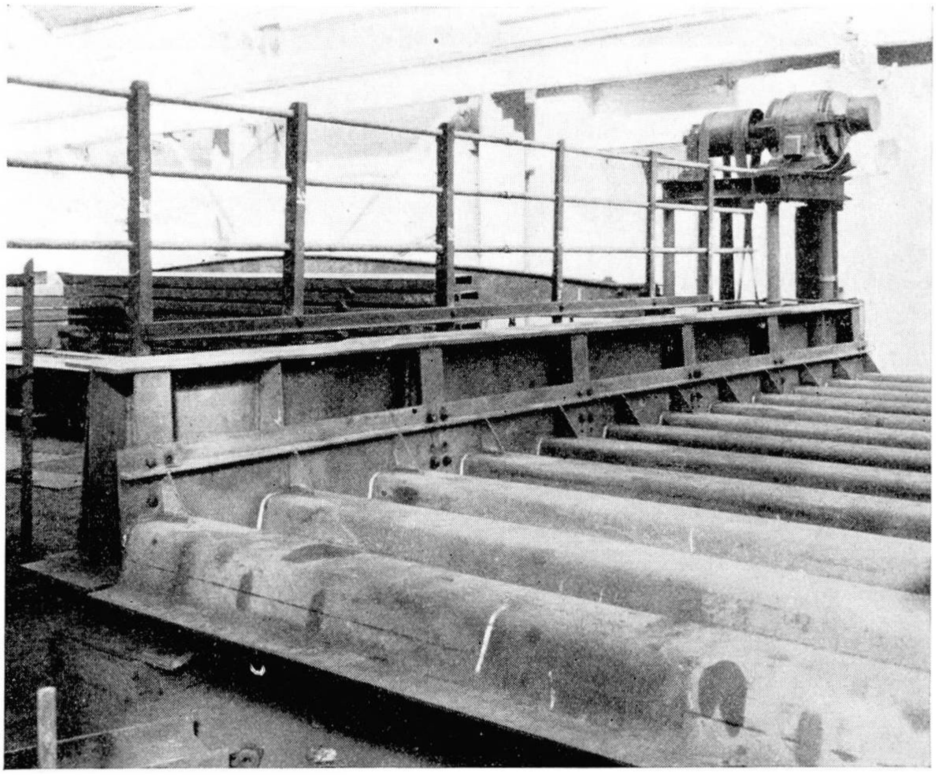


Fig. 11. Transverse pressed steel troughing welded to ballast plates which are bolted to the inclined faces of the stiffeners on a welded plate girder

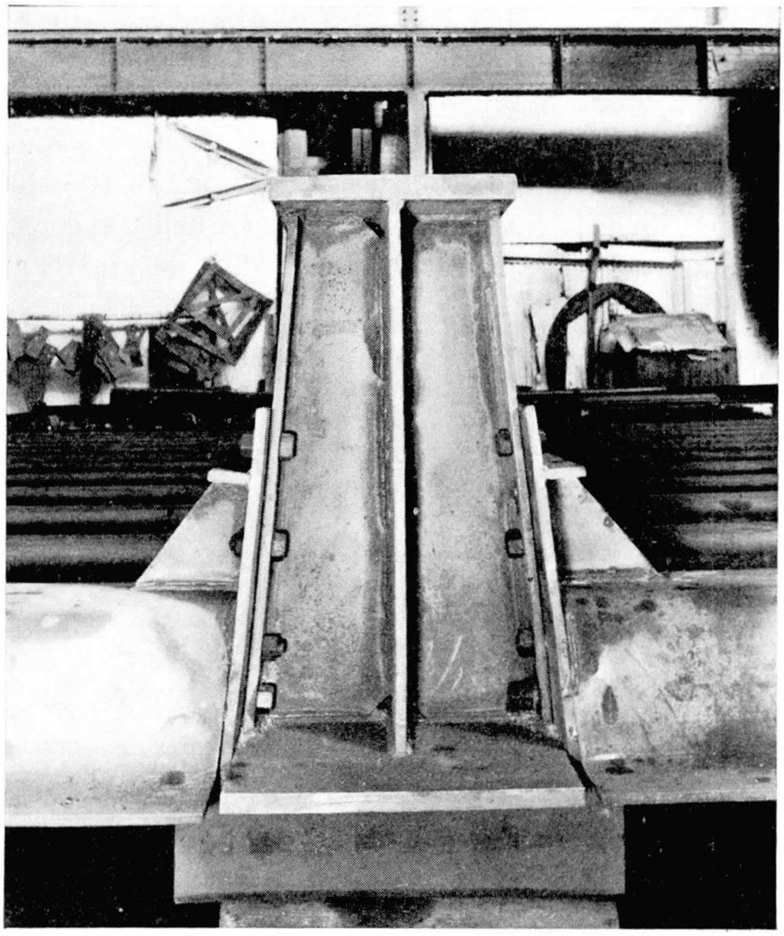


Fig. 12. The bridge shown in fig. 11. Details of the bolted connections of the ballast plates to an intermediate main girder

Fig. 13. Transverse pressed steel troughing connected to a main girder by cleats at the top and by plug-welds at the bottom

has been welded to side ballast plates which are in turn site-bolted to inclined stiffeners on the main girders. In this case a much greater degree of end-fixity is provided for and movement between the decking and the main girders has been eliminated.

Where transverse troughing rests on the bottom flanges of the main girders an improvement has been made by plug-welding the bottoms of the troughs to the packings, the packings being either shop-welded or countersunk-riveted to the flanges (fig. 13). The plug-welds are of course made at site.

Transverse Beams and Reinforced Concrete Cast in Situ

When conditions permit the new superstructure to be rolled in, a decking of transverse steel joists at about 2′–8″ centres, with concrete deck reinforced with rods passing through the webs of the joists, is often used. But again, the details of the end fixing of the joists to the main girders requires special attention. Sometimes the underside of the concrete is cast flush with the bottom flanges of the joists, and although no account is taken of the strength of the concrete a higher working stress is permitted in the joists. Leaving the underside of the joists exposed does of course increase the area to be painted. The alternative is to encase the bottom flanges of the joists in concrete, and

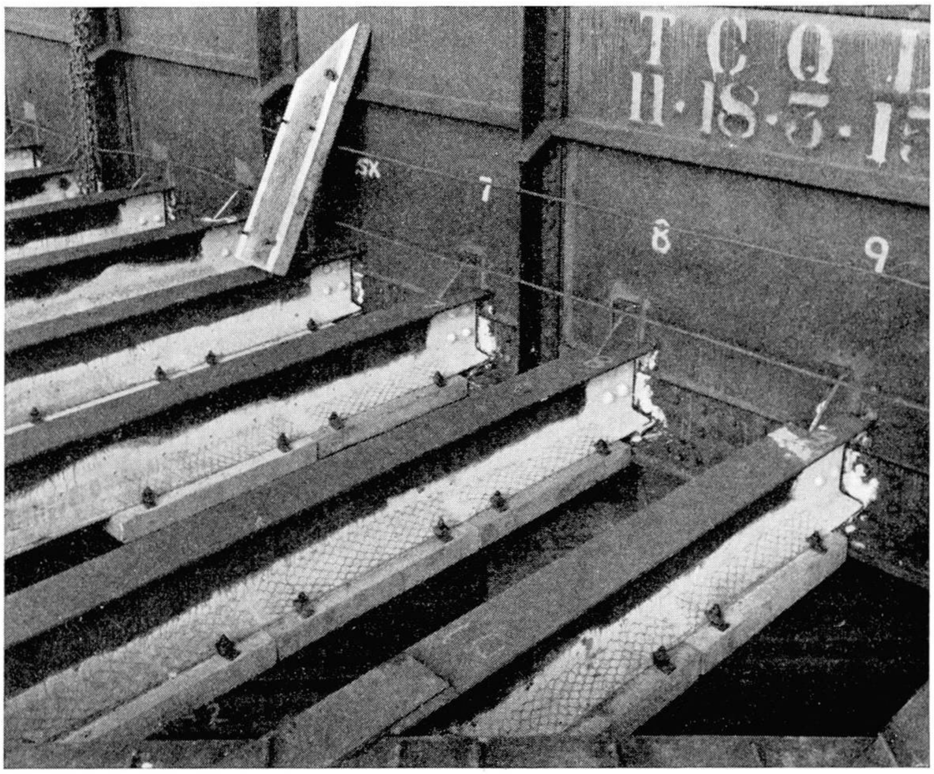


Fig. 14. Transverse steel joists with precast concrete slabs protecting the bottom flanges

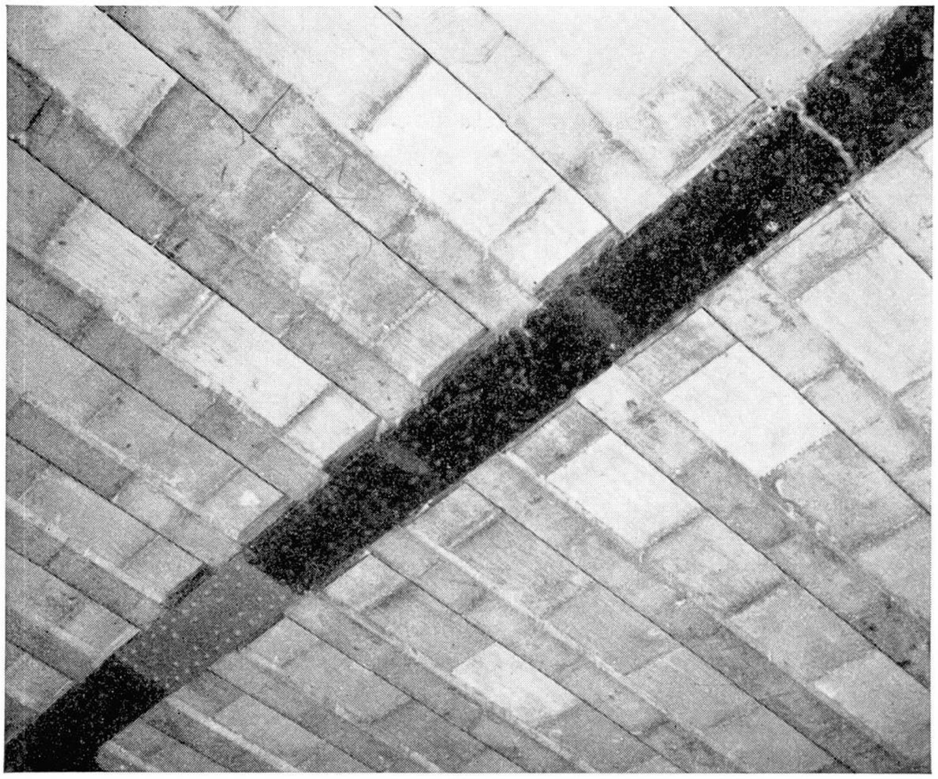


Fig. 15. The completed underside of the bridge shown in fig. 14. The gaps between the joists are spanned with precast concrete slabs forming the permanent shuttering supporting the situ-cast concrete of the deck

this is sometimes done with precast components secured in place with U-bolts and clips. Precast concrete slabs between the joists act as permanent shuttering for the situ-cast reinforced concrete as shown in figs. 14 and 15.

Primary and Secondary Decking Units

Where the span cannot be completed before rolling in and where the erection is to be done piecemeal much time can be saved by the adoption of a decking of primary and secondary units. Each primary unit consists of a pair of steel cross girders connected by steel stringers and the whole encased in concrete, constituting a reinforced concrete deck slab (fig. 16). The cross girders are spaced along the span so as to maintain six-foot intervals between the cross girders, and the space between the primary units is spanned with multiple precast reinforced concrete secondary units. The cross girders in the primary units are connected to the inclined faces of the stiffeners on the main girders by high-strength bolts, the joints being so arranged that the ends of the cross girders rest on shear plates. This type of construction can be assembled relatively quickly, there being no site riveting or site welding, but of course the top of the deck has to be protected with overall waterproofing.

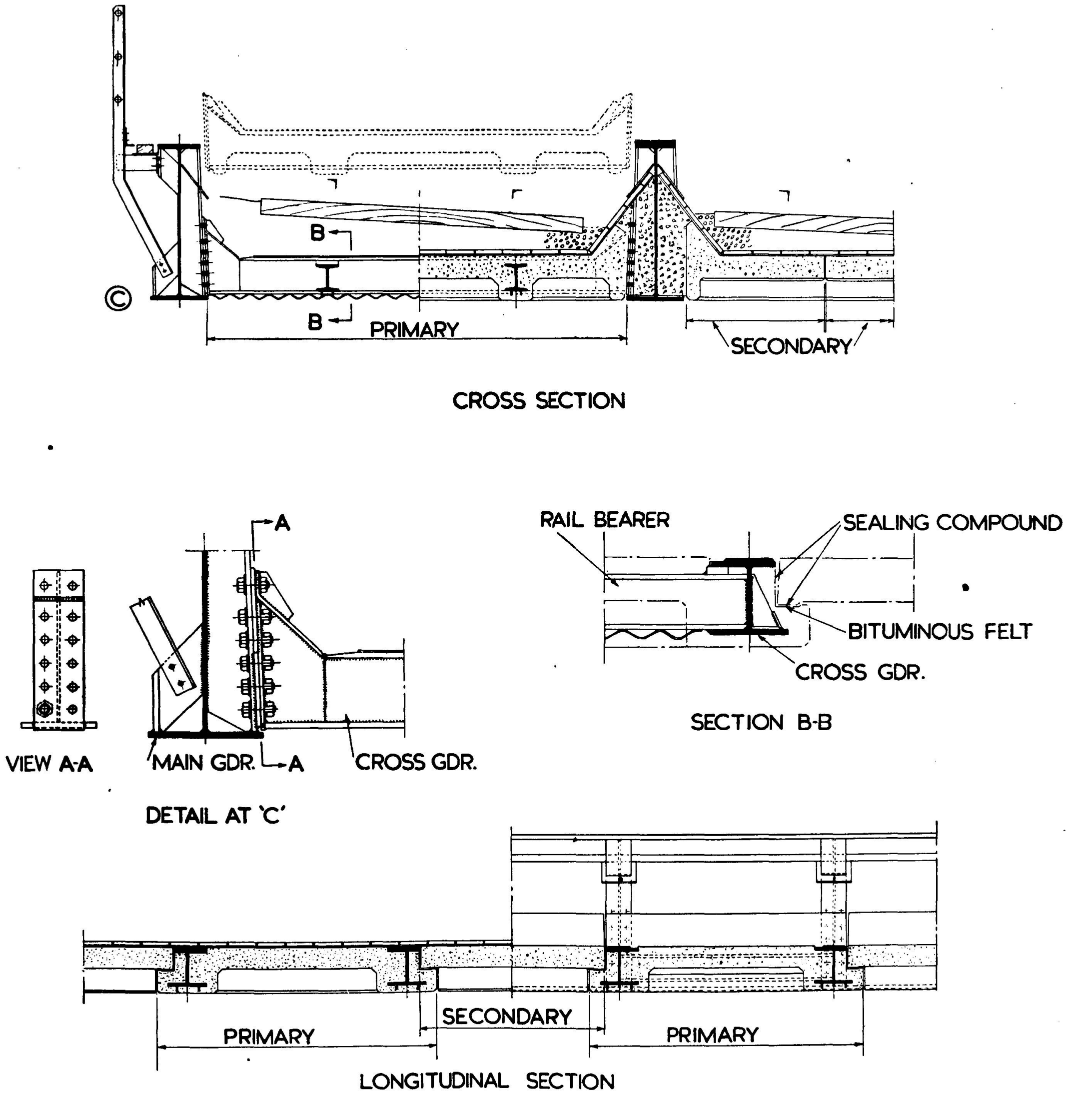


Fig. 16

Steel Cross Girders and Precast Concrete Jack Arches

There is no doubt that within reasonable limits cross girders should be as deep as possible. Where construction depth permits, deep cross girders have the bottom flanges encased in concrete shaped to form skewbacks for precast reinforced concrete jack arch units (fig. 17). With deep cross girders it is relatively easy to provide full end-fixity at the end connections to the main girders. Precast reinforced concrete ballast units bolted against the stiffeners provide haunching against which the waterproofing is finished off, and the space between the ballast units and the main girders is easily filled in with situ concrete which can be placed after the bridge has been opened to traffic. This form of construction becomes somewhat complicated in the case of skew spans, and when the angle of skew is very acute the trimmer girders become long and heavy.

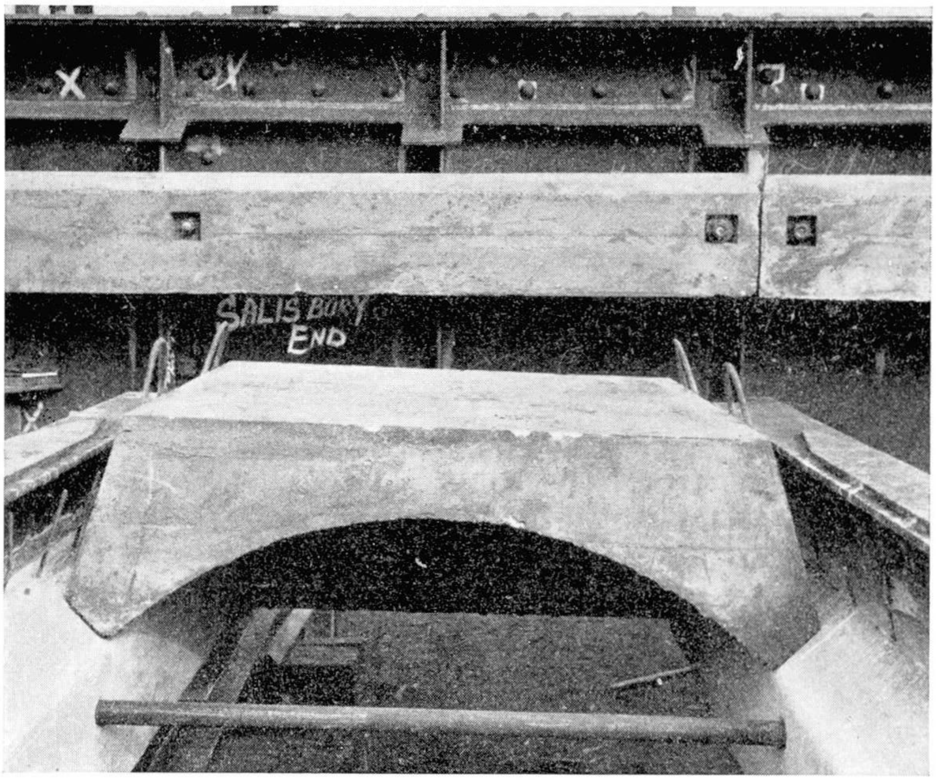


Fig. 17. A precast reinforced concrete jack arch resting on the concrete skewbacks precast on the bottom flanges of the cross girders

In modern practice it is desirable that the whole superstructure of each span shall be supported independently of the abutments, except through the bearings of the main girders.

Prestressing Concrete encasing Steel Cross Girders

The encasing of the undersides of cross girders in concrete is open to objection because when the cross girder deflects the bottom surface of the

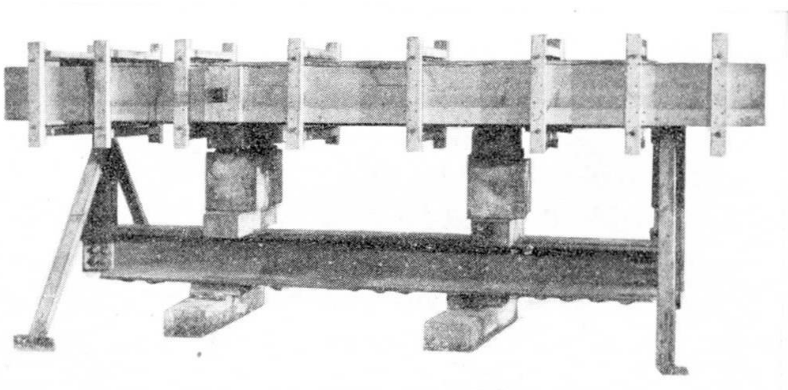


Fig. 18. A pair of steel cross girders held apart in a condition of prestress while the concrete is being cast around the upper girder

concrete must be stretched in tension. Cracking is to be expected in the concrete and the encased steel ceases to be completely protected. With the object of overcoming this trouble, cross girders can be prestressed. They are bent to their fully deflected shape while the encasing concrete is cast around them; and upon release the concrete remains permanently in compression, the bottom fibre stress reducing to zero but not changing to tension under full live load. Fig. 18 shows a pair of cross girders strained while the upper one is being encased in concrete.

Prestressed Concrete Deck Units

The latest development in the provision of large decking units, is the deck consisting of prestressed concrete units spanning between the main girders

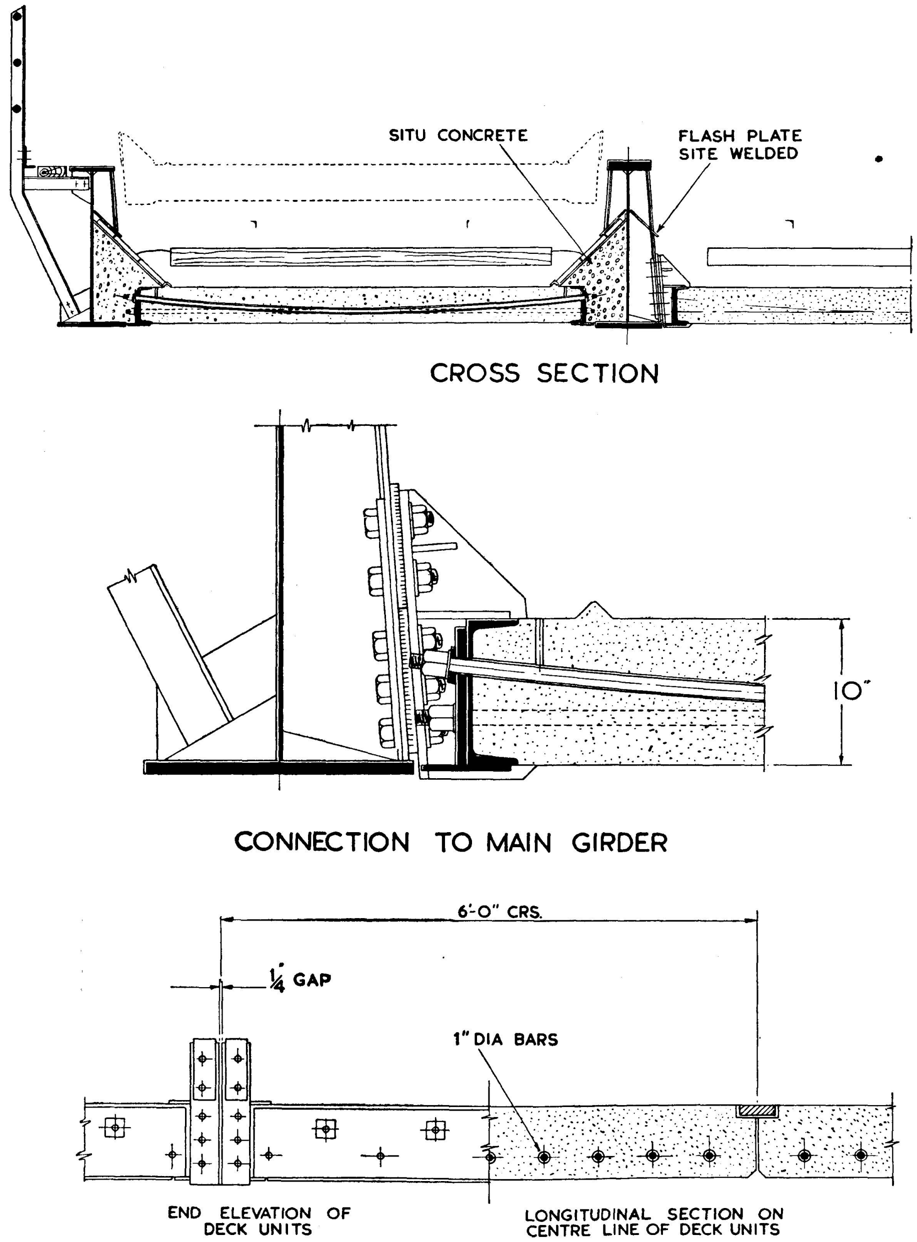


Fig. 19

and measuring about 6 feet in width along the line of the span (fig. 19). The ends (adjacent to the main girders) of these precast units are bounded with mild steel channels strengthened with thick plates which form the end shuttering during casting and are the anchorages against which the nuts of the Lee-McCall prestressing bars bear. With the main girders at about 12 feet centres, some nine 1-inch diameter bars are used. Connection to the stiffeners of the main girders is by high-strength bolts passing through angles welded to the channels and through the inclined flanges of the stiffeners. Shear plates, on which the deck units land, are designed to take the full end shear from the units. The bolted connections to the main girders are designed to develop the full end-fixity of each unit. The minimum thickness of the deck units is 10 inches and as waterproofing is not required over the prestressed concrete except at the joints, a relatively shallow construction depth is obtained. Each prestressed concrete unit is self-contained and there is no transverse post-tensioning of the units either individually or as a whole.

The End-fixity Conditions of Deck Units and Cross Girders

Reference has been made to the importance of developing the necessary strength of the connections between the decking and the main girders in a through-type span. It is not easy to provide simple supports and the decking in a double track 3-girder span is considered as being continuous between the outer main girders. Apart from the full end-fixity which must obviously be provided at the connections to the intermediate main girder, some degree of end-fixity must also be provided at the connections to the outer main girders because the deflection of the decking units or cross girders necessarily induces a change of slope at the ends of these units or girders. A note on the design of a prestressed concrete deck with its connections to the main girders is given in an appendix to this paper.

High-Strength Bolts for Permanent Site Connections

Reference has been made to the use of high-strength bolts for field connections. These bolts have case-hardened washers under heads and nuts, and comply with the following specification:

Materials

Bolts are to be of steel, heat treated to give a minimum ultimate tensile strength of 47 tons per sq. in. with an elongation of not less than 20 per cent on a B.S. test piece C, and a minimum yield strength of 38 tons per sq. in.

Nuts are to be of mild steel complying in all respects with the requirements of B.S. 15: 1948.

Washers are to be of steel, hardened by carburizing (not cyanide) to a minimum depth of 0.015 inch and to a hardness of 65—70 Rockwell, A Scale.

Dimensions and Finish

Bolts, nuts and washers are to be to the dimensions specified in B.S. 916: 1946 for Black Bolts and Nuts, except that washers for $^3/_4''$ and $^7/_8''$ dia. bolts are to be 7 S.W.G. (0.176 in.) thick. Heads and nuts of bolts are to be hexagonal.

Bolt heads and nuts are to be machined on the bearing surfaces and care shall be taken that these surfaces are truly square to the axis of the thread. The radius of the fillet between the head and shank shall be made $^1/_{32}$ inch.

Washers are to be machined or planished flat.

Threads are to be of B.S.W. proportions, to the limits and tolerances for "Medium Fit" specified in B.S. 84: 1940.

The shanks of the bolts need not be machined, since the bolt holes are greater in diameter than the bolts.

The bolts are tightened to a predetermined torque in accordance with the following table:

Diameter of bolt (in.)	$^3/_4$	$^7/_8$	1	$1^1/_8$
Torque (lbs. ft.)	304	489	734	1040

It is claimed then when so tightened, the bolts, stressed to within 85% of the yield point of the steel, will stay tight under exacting conditions[4]).

The Choice of Welding or Riveting for Fabrication

While savings ranging between 13 and 21 per cent in weight and between 16 and 27 per cent in the cost of girderwork have been achieved when adopting arc welding instead of riveting, there is as yet no complete changeover to welding for new bridge girderwork. Not all the fabricators in Britain are yet properly equipped for welding and it is not always possible to obtain favourable quotations for welded work. Also there still persists a lack of confidence in the quality of the workmanship in a craft in which supervision is so very much more important than it is in riveting, where testing after fabrication is so simple and so certain. Nevertheless welding is coming into its own; underline bridges with the main girders of lengths up to 110 ft. are under fabrication. Such girders are designed to facilitate welding by an automatic process. In a contract for the supply of a number of girders, the greatest possible use is made of standardisation so far as depth of girder is concerned, as was the case in the Scottish bridges.

[4]) Ctee Rep. "Iron and Steel Structures", Amer. Rly. Engng. Assn., vol. 51, No. 485, p. 441 (Jan. 1950).

The Surface Preparation of New Steelwork

Much attention is now paid to the removal of mill scale and the preparation of the surface of new steelwork. Fabricators when tendering for the supply of new girderwork are asked to state the method they propose to use to remove mill scale. The choice usually rests between:

a) the pickling process whereby the steel, before fabrication, is immersed in a bath of hydrochloric acid (one part of acid to 19 parts of water) and thoroughly scrubbed and washed with clean water before being dried and given the first priming coat of red lead paint before rust is allowed to form;
b) flame-cleaning after fabrication;
c) shot-blasting after fabrication;
d) controlled weathering.

Whichever process is used, it is now usual to call for two priming coats of paint (namely one of red lead and one of one-third red lead and two-thirds white lead), as well as a first protective undercoat of micaceous iron ore paint, to be applied to all exposed surfaces before delivery from the shop. In addition to providing protection against rusting, this has the advantage that, apart from patch-painting to make good abrasions in transit, the painting in the field is confined to the application of a fourth and finishing coat of micaceous iron ore paint. When the mill scale is removed by the pickling process, those surfaces which will ultimately be encased in concrete are given temporary protection by the application of a cement wash.

The advantage of proper surface preparation of exposed steelwork is manifest in the improved life of the paint. Under normal conditions a period of up to 11 years is expected before repainting becomes necessary — this being at the beginning of the breakdown of the protective coatings and *before* the priming coats of paint show signs of breakdown.

Reconstruction Erection Schemes

In considering the various methods of erection, it is necessary to remember that in addition to the periods of total occupation required for assembly and perhaps rolling-in, a speed restriction has to be enforced from the commencement of the work of preparing the old superstructure for removal until the new span is in position and the ballasted permanent way is restored. From the traffic point of view it is essential to keep this period, during which the speed restriction must be in force, to a minimum. It is often preferable to increase the number of complete occupations if this results in a reduction of time during which the speed restriction must be in force. Fig. 20 gives some indication of the number of occupations and the period of speed restrictions in relation to the method of erection chosen for different types of construction for a double track 3-girder 40 ft. span underbridge. For the purpose of this

OCCUPATIONS AND SPEED RESTRICTIONS FOR THE RECONSTRUCTION OF A DOUBLE TRACK 3-GIRDER 40FT THROUGH TYPE SPAN.

SITE WORK REQUIRED FOR REMOVAL OF EXISTING SUPERSTRUCTURE, RENEWAL OF BEDSTONES, ERECTION OF NEW SPAN, WATERPROOFING AND RELAYING BALLASTED PERMANENT WAY.

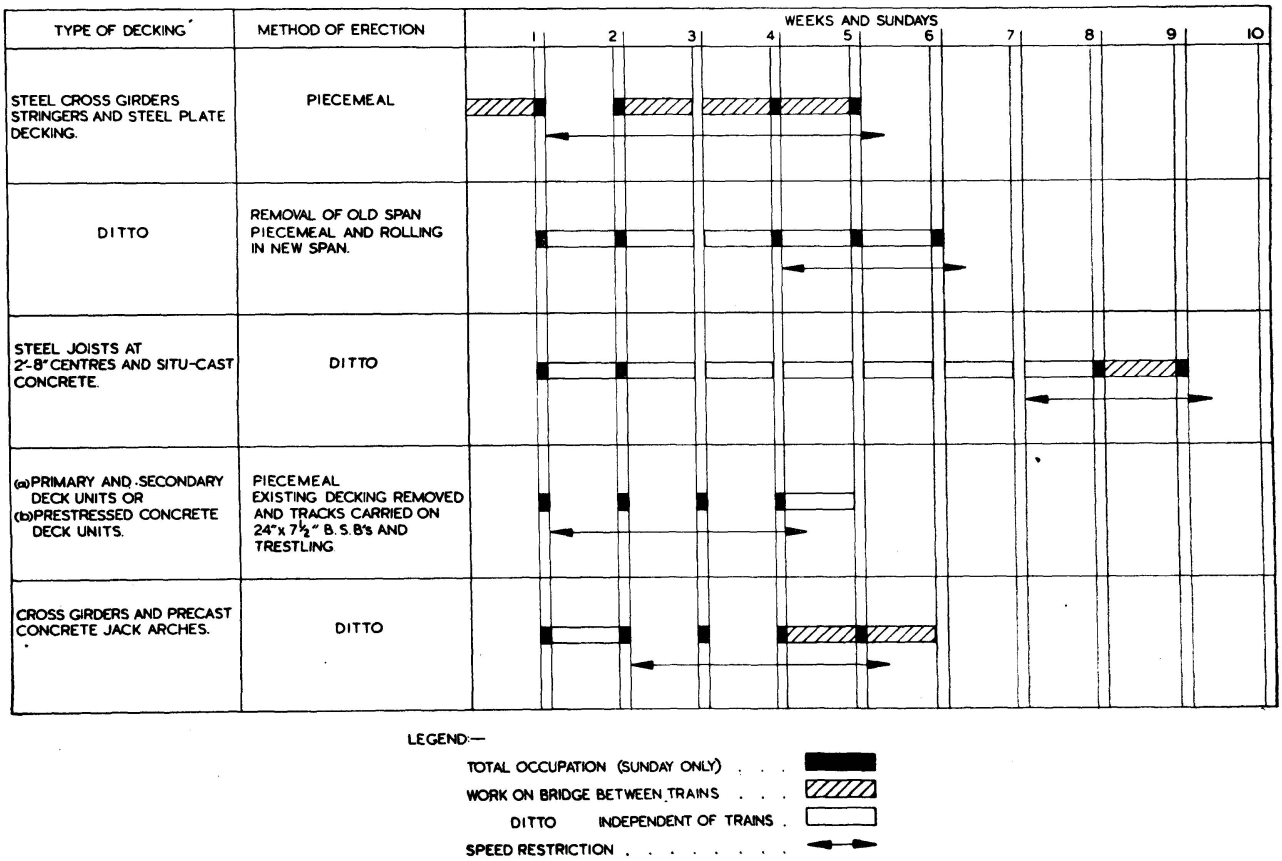


Fig. 20

chart, it has been assumed that total occupations can be given only on Sundays. Naturally, conditions vary greatly at different sites and it may be possible to permit a restricted train service on at least one track throughout week-end occupations. In assessing the cost of erection it is not usual to include the extra cost of fuel, time, etc. involved in slowing down and accelerating trains due to a speed restriction, but obviously this may amount to a considerable sum on a busy line and the need for keeping the period of the speed restriction to the minimum is very real.

Acknowledgements

The Author is grateful to Mr. M. G. R. Smith, M.B.E., Regional Civil Engineer, Western Region, British Railways, for permission to present this Paper, and to Mr. F. M. Easton, A.M.I.C.E. for the note given in the appendix. He also expresses his thanks to those members of his staff who helped in collecting data and preparing diagrams.

Appendix

Prestressed Concrete Deck Slab and its Connections to Main Girders

Type. As shown in Fig. 19.

Dimensions. Main girders 12′ 10½″ c. to c. Slabs, 10′ 10½″ span over end channels, width 6′ 4″, depth 10″. Prestressed by 10 bars, 1″ diameter with centres 3½″ above bottom of slab.

Loads. Self weight of slab, 9,200 lb. Ballast and permanent way, 5,100 lb. Live load 37 tons on two wheels at 5 ft. centres. The distribution of the live load, through rails, sleepers and ballast, will be nearly uniform; to facilitate calculations, an equivalent uniformly distributed live load of 41.7 tons is taken.

Prestress. The initial prestress in the bars is 42 tons/sq. in., the effective prestress is 35 tons/sq. in.

Bending Moment and Deflection. The bending moments acting on the slab, and the consequent deflections, are shown for successive stages of loading in the following Table. In calculating deflections, Young's modulus for concrete, E_c, is taken as 5×10^6 lb./sq. in.

Deflection of Slab and Slope at Inner End

Loading due to:	Moment at centre	Deflection at centre	Slope at inner end	Moment at inner end
	lb ins	ins	radian	lb ins
a) Self weight	149,800	.0084	.00021	0
b) Prestressing	−924,000	−.0511	−.00125	0
a) + b)		−.0427	−.00104	0
c) Bolting up ends	382,400	.0256	+.00104	761,600
a) + b) + c)		−.0171	0	761,600
d) Placing ballast and per. way	41,600	.0019		− 83,200
a) + b) + c) + d)		−.0152	0	678,400
e) Live load	762,500	.0342		−1,525,000
a) + b) + c) + d) + e)		.0190	0	−846,600

Under the prestressing forces, the slab has an upward deflection before being placed in position. The corresponding slope at the ends of the slab is not allowed for in constructing the steelwork, and in consequence there is a tapered opening, calculated at 0.00104 radian (i_A), which must be closed when bolting the slab in place. At the connection to the outer main girder there will be negligible resistance to closing the gap; but at the centre girder, tightening

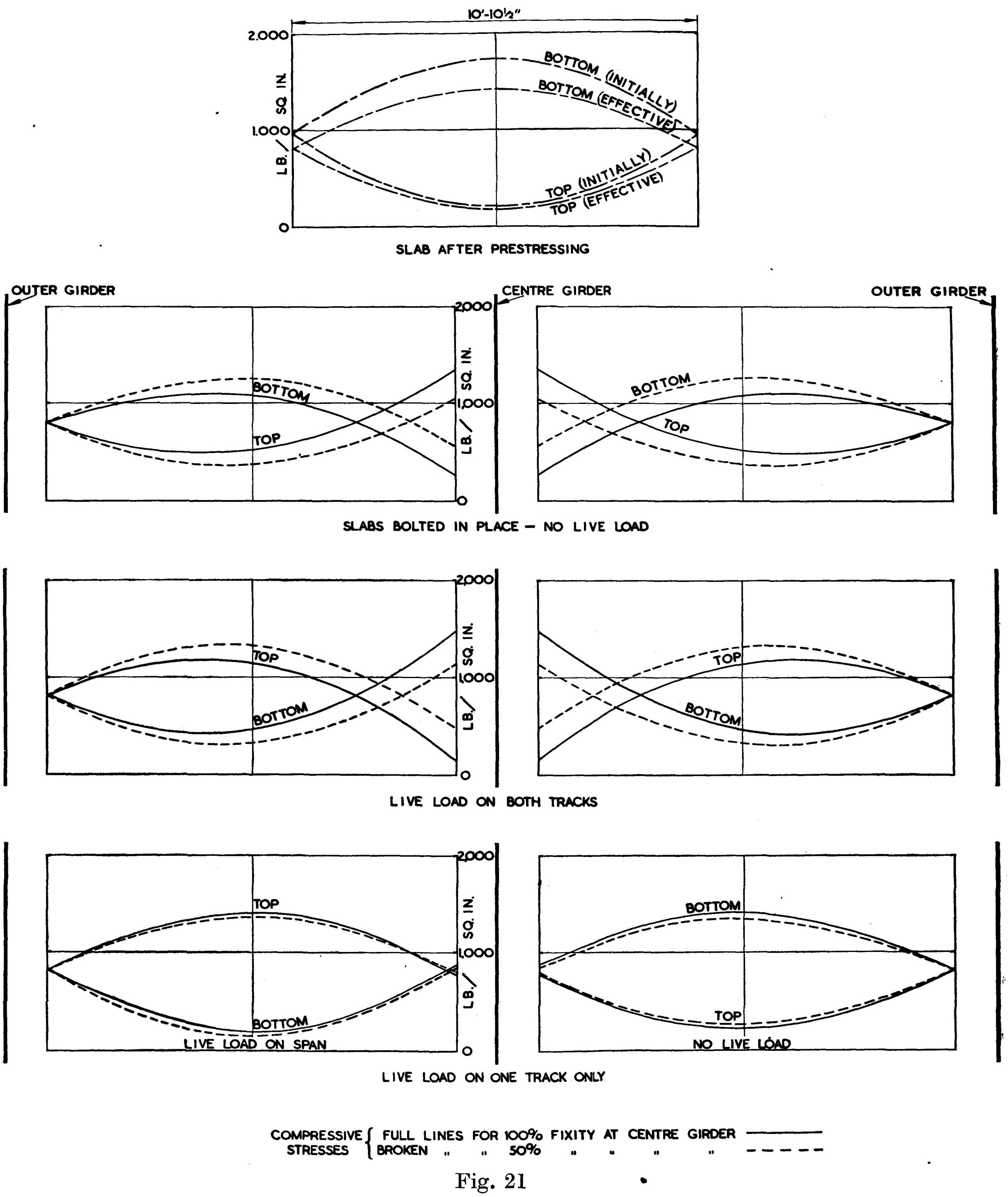


Fig. 21

the bolts to close the gap will impose a bending moment on the slab, equal to $3\,E_c\,I\,i_A/l$, which works out to 761,600 lb. ins. Placing the ballast and permanent way will reduce the end moment to 678,400 lb. ins.

The live load will be borne by the slab acting as a beam with the inner end fixed. The end moment due to the live load, being opposite in sense to the moment with no live load, will leave a resultant of $-846{,}600$ lb. ins. to be borne by the connection to the main girder.

The end moments shown in the Table have been calculated on the assumption of full fixity at the centre girder. As the structure cannot be perfectly rigid in a transverse direction, the actual end moments are likely to be of smaller amount than those calculated.

No account has been taken of the effect of unequal deflections of the outer and centre main girders. There would be some inequality in the loading, due to the fact that a positive moment exerted by the bolted connection at the centre girder must be accompanied by a relief of shear at that point, and a negative moment by increase of shear. The transfer of load when bolting up the slabs will partly balance the increase of live load shear at the centre girder due to fixity. In designing the main girders, full or partial allowance for these effects may be made, if desired.

Strength of Bolted Joint. Details of the end connections are shown in fig. 19. Positive bending moments will be associated with maximum tension on the lower bolts of the joint and negative bending moments with maximum tension on the upper bolts. The moment of greatest numerical magnitude is $-846{,}000$ lb. ins. It is carried by two rows of 5 bolts each, at an average pitch of $3\frac{5}{8}$ ins. Assuming the compressive reaction to the tensile loading, under the bending moment, to coincide with the centre line of the lowermost bolts, the tension on the uppermost bolt will be 15,570 lb. = 6.95 tons. The bolts are $1\frac{1}{8}''$ diameter with B.S. Whitworth threads, and the stress for this load, on mean thread area, is 9.07 tons/sq. in., well within the capacity of the high-strength bolts used for the work. The initial tightening brings tension upon the bolts much in excess of the load due to bending moment, so that there is no separation of the contact surfaces.

Stresses in Concrete Slab. The calculated stresses in the slab, at successive stages of loading, are shown in fig. 20 both for full fixity and for half fixity of the connections to the centre girder.

Summary

The paper deals with modern practice in the design of underline girder bridges on British Railways, Western Region.

Design values of live load, including allowances for impact effect due to hammer blow and rail joint, and also for lurching and for the longitudinal distribution of loading through the track, are defined.

The advantages of continuing the normal cross-sleeper ballasted track across the bridge are mentioned and the waterproofing of the bridge deck is described.

Comparative costs of various types of construction, for a double-track, 40-ft. span, are given.

The forms of construction used for modern deck-type and through-type plate girder spans are described in detail, with a discussion of the comparative merits of various types of decking, including those in which concrete is used, either in-situ, precast, reinforced or prestressed.

Mention is made of some features of riveted construction which have been found to give trouble in maintenance, through loosening, corrosion or lack of watertightness; and the measures now taken, to avoid or overcome the difficulties, are described.

A description is given of some modern design features which have the object of facilitating the erection of bridges on site, and the merits of various alternatives are compared with respect to the necessary periods of occupation and the speed restrictions involved.

The use of high-strength bolts for field connections, following the experience of American railway engineers, is commended.

The choice between welding and riveting for fabrication is touched upon, and reference is made to current practice in surface preparation and painting of steelwork.

Zusammenfassung

Der Aufsatz behandelt die neuesten Erfahrungen in der Berechnung von Trägerbrücken für Unterführungen bei den britischen Eisenbahnen der westlichen Region.

Für die Nutzlasten einschließlich den dynamischen Wirkungen durch Schläge, Schienenstöße und infolge Schlingerns und für die Lastverteilung in Längsrichtung durch das Geleise werden Rechnungswerte definiert.

Die Vorteile des über die Brücke durchgeführten Schotterbettes mit normalen Querschwellen und Geleisen werden dargestellt und die Dichtung der Brückendecke wird beschrieben.

Für eine zweigleisige Brücke von 40′ (12 m) Spannweite werden für die verschiedenen Typen Kostenvergleiche angestellt.

Die bei modernen vollwandigen Balkenbrücken mit oben oder unten liegender Fahrbahn verwendeten Konstruktionsarten werden bis in Einzelheiten beschrieben und die Vorzüge der verschiedenen Fahrbahnarten einschließlich denjenigen aus an Ort und Stelle hergestelltem oder vorfabriziertem Beton mit gewöhnlichen oder vorgespannten Armierungen werden verglichen.

Es werden einige Charakteristiken der genieteten Konstruktionen erwähnt, die im Unterhalt zu Schwierigkeiten Anlaß gaben (Lösen der Nieten, Korrosion oder mangelnde Dichtigkeit). Die Maßnahmen, die ergriffen werden, um diese Schwierigkeiten zu vermeiden oder zu überwinden, werden beschrieben.

Es werden einige moderne Entwurfsformen angegeben, die den Zweck haben, die Montage der Brücken zu erleichtern und die Vorzüge der verschie-

denen Möglichkeiten werden verglichen bezüglich der notwendigen Beschäftigungszeiten und der Geschwindigkeitsbeschränkungen.

Auf Grund der Erfahrungen amerikanischer Eisenbahningenieure wird die Verwendung hochwertiger Bolzen für Montagestöße empfohlen.

Die Frage der Wahl zwischen geschweißter und genieteter Ausführung wird aufgeworfen, und es wird auf die allgemein üblichen Verfahren der Oberflächenbehandlung und des Anstrichs von Stahlkonstruktionen hingewiesen.

Résumé

L'auteur expose la technique moderne en matière de calcul des ponts à poutres pour passage inférieur, sur le réseau des chemins de fer britanniques, dans la région Ouest. Il définit les valeurs de calcul de la charge variable, en tenant compte des tolérances correspondant à l'effet d'impact dû aux à-coups et aux joints des rails, ainsi que de la répartition longitudinale de la charge sur la voie.

L'auteur mentionne les avantages que l'on peut trouver à adopter également, sur le pont lui-même, la disposition avec ballast et traverses et à assurer l'étanchéité du tablier.

Il indique les prix comparés de différents types de constructions, pour une voie double et pour une portée de 40′ (12 mètres).

Les différents modes de construction utilisés pour les ponts modernes avec poutres à âme pleine, des types à tablier supérieur et à tablier inférieur, sont exposés d'une manière détaillée avec étude des avantages comparés des différents types, y compris ceux qui impliquent l'emploi du béton avec mise en œuvre sur le chantier, prémoulage, armature ordinaire ou précontrainte.

Mention est faite de différentes caractéristiques de la construction rivée, dont on a constaté qu'elle donnait lieu à des difficultés d'exploitation par suite du desserrage, de la corrosion ou du défaut d'étanchéité des rivets; l'auteur indique les dispositions qui sont actuellement prises pour éviter ou surmonter ces difficultés.

Il décrit certaines caractéristiques modernes de conceptions qui ont pour but de faciliter le montage des ponts sur place et compare les avantages de différentes solutions, du point de vue des périodes d'occupation à prévoir et des restrictions à imposer à la vitesse.

L'emploi des boulons à haute résistance pour les assemblages sur place est étudié à la lumière de l'expérience acquise par les ingénieurs des chemins de fer américains.

L'auteur aborde la question du choix entre la construction soudée et la construction rivée et fait mention de la pratique courante en matière de préparation et de peinture des charpentes métalliques.