Recorded failures of electrically welded wrought-iron and mild steel bridges

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RECORDED FAILURES OF ELECTRICALLY WELDED WROUGHT-IRON AND MILD STEEL BRIDGES.

DÉTÉRIORATIONS DANS LES PONTS DE FER ET D'ACIER SOUDÉS ÉLECTRIQUEMENT.

BRUCHERSCHEINUNGEN AN ELEKTRISCH GESCHWEISSTEN SCHMIEDEEISEN- UND FLUSSTAHLBRÜCKEN.

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The method of repairing and strengthening wrought iron and mild steel structures by means of electric welding on the London and North Eastern Railway was first adopted by Mr. John Miller, Engineer for the North Eastern Area of the London and North Eastern Railway, in 1927 and the method has since been made use of almost exclusively for this class of work and in some degree for the construction of new structures. That it has not been more generally adopted for new construction is because the costs at the present time for this class of work are somewhat more costly than riveted work.

Owing to the meagre data available on which to base calculations of strength and for technique in carrying out welding on different kinds of materials in different positions and under varying conditions, Mr. Miller decided not hastily to discard the old established practices and the vast experience gained in connection with riveted work, but to advance slowly after having established his own data as regards the strength of welded connections and so to design the details of repair, strengthening and new work, that failure of the welding at any point would not be a matter of serious moment.

A great number of tests and experiments have been carried out, not only for the purpose of ascertaining the strength of welds and weld metal, but also to analyse and determine the little known new problems of heat and shrinkage stresses and their relation to the strength of the structure and the measures which would be required to be taken to counteract or neutralise their presence. An obstacle in pursuing this rather large subject of investigation lay in the fact that no experienced or efficient welders were originally available to whom the work could safely be entrusted. As data for calculation were established and experience gained, electric welding was applied to all classes of new and old structural work, such as girders, frame work of buildings, roofs, crane gantries etc. Likewise repair and strengthening work; for instance, replacement of corroded girder webs, flanges, diagonals and verticals of plate and lattice girders as well as strengthening of weak parts of same.

After eight years, during which period a great number of various kinds of welded structural work have been carried out, a considerable fund of knowledge has been accumulated. Experiences of failures must be reckoned as not the least instructive and valuable, as they are the finger posts indicating that along certain roads efficiency cannot be attained. It occurred to the writer, who from the outset was detailed by Mr. Miller to carry out the tests and experiments in regard to welding and to prepare the drawings and supervise the carrying out of the work, that some account of the experience which has been derived from the failures might be of more than general interest and in this view his Chief concurs. The writer wishes however to point out, that the views expressed by him in this paper represent his own views and ideas.

Although the total amount of welding work carried out by Mr. Miller's staff or carried out under contract is very great, the actual cases of failure are relatively small.

The causes of failures may be grouped under three headings:

1. Welding Stresses.
2. Shock.
3. Fatigue.

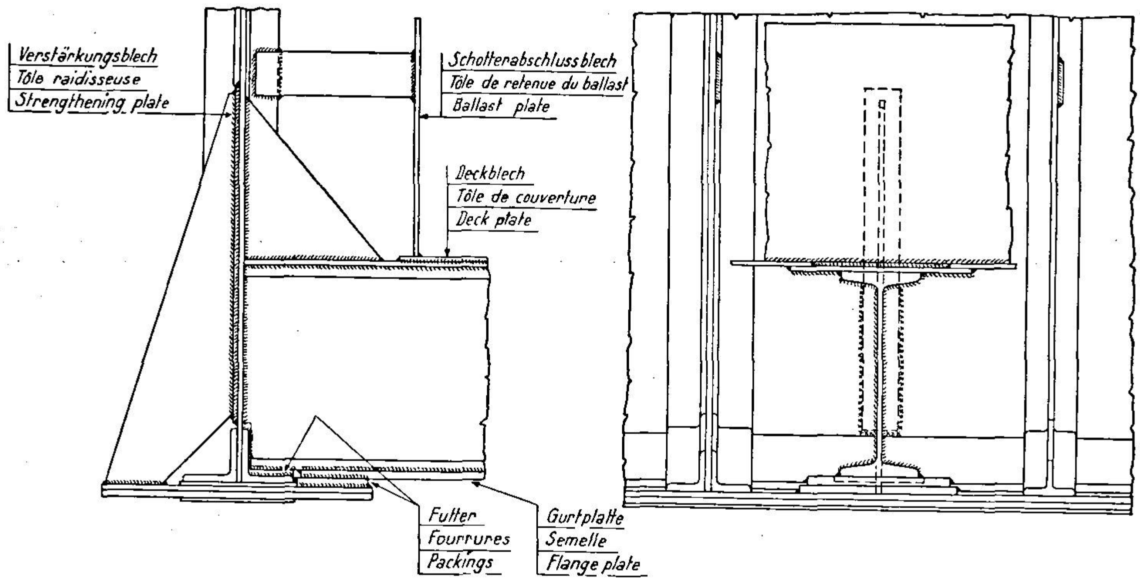

Fig. 1.

Einzelheiten von Knotenblech- und Stegblechaussteifungen (Brücke No. 32, South shields Linie).
Détails des goussets et raidisseurs de l'âme. (Pont n° 32, ligne South shields).
Details of gussets and web stiffeners. (Bridge N° 32, South shields branch).

In a sense all three causes might be embraced in reality under one only: "Insufficient provision of material for withstanding the strains imposed on the structure by external loads".

Welding Stresses.

Welding stresses which are set up by the contraction of the weld metal itself cause distortion; indirectly the heating up of the welded parts sets up contraction stresses inasmuch as in the heated condition the parts to be joined become rigidly fixed and when cooling down they may contract unequally and thus also cause distortion. When such jointed parts, which are in a state of internal stress, are subjected to exterior forces which produce bending, tensile or compressive stresses, previously highly stressed welded parts may become overloaded to the extent of actual fracture.

The first experience of this type of failure occurred in the case of the reconditioning of a large plate main girder of a wrought iron bridge the floor of which was entirely renewed. As it was necessary to place the new cross girders between the old cross girders, where there were no web stiffeners, provision had to be made at these points for proper distribution of end fixing moments and shear from the cross girder ends. A strengthening plate was therefore welded to the outside of the web of the main girders and a triangular gusset plate welded on to it directly opposite the new cross girder end, as shown on Fig. 1.

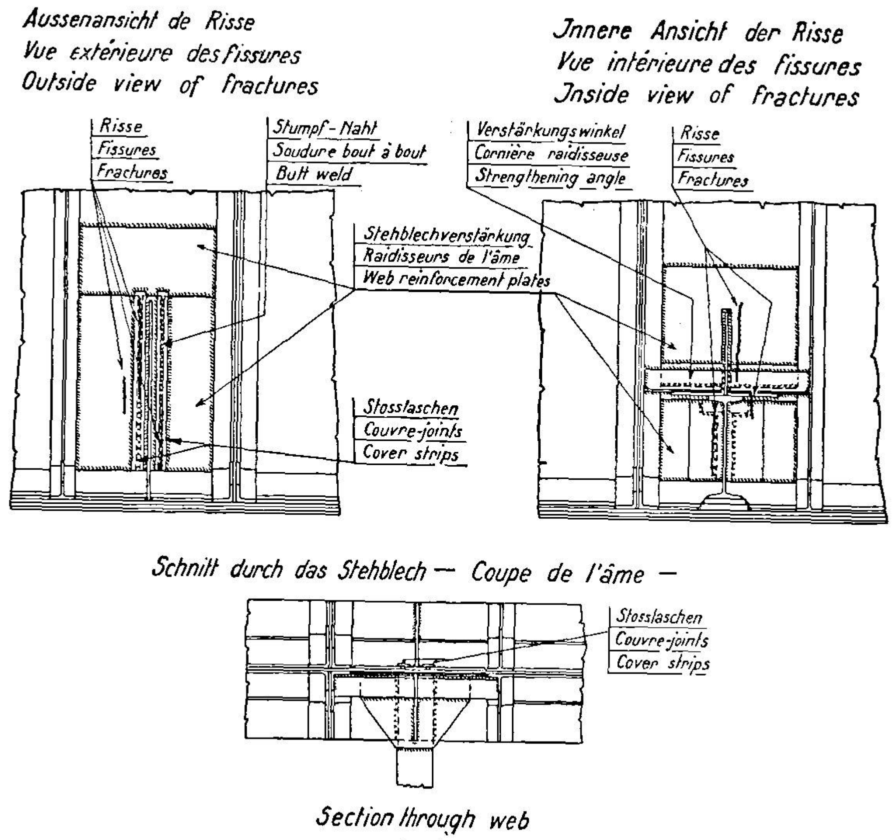

Fig. 2.
Einzelheiten eines reparierten gerissenen Stegblechfeldes.
Détail d'un champ réparé de l'âme où celle-ci était fissurée.
Details of repaired panel of fractured web.

The webs of the main girders were badly corroded and in places reduced down to 1/16″ and less in thickness. On welding the flat strengthening plates to the main girder web plates, fractures developed in two instances, which travelled up the web plate at the same rate as the weld was run. As the crack formed while the weld metal was being deposited and not during the cooling down of the heated part of the girder, it is reasonable to assume that the stress set up by the contraction of the weld metal added to the existing shear stress brought the total stress up beyond the ultimate strength of the web plate. Fig. 2 shows the repairs.

Extra web cover plates were therefore tack welded to the "T", angle stiffener and strengthening plate, the fracture having been "V' d" out and butt welded previously. The new web reinforcement plates were then finally butt welded to the strengthening plate and the toes of the "*T*" and angle stiffeners of the girder.

In the case of repair work like this, where the webs are thin, web strengthening plates are now always provided, which are welded to the adjoining main angles and stiffeners.

In another instance where the webs of the main girders had been holed through corrosion and the webs required to be repaired, fractures took place along the welded seams which were probably due to the contraction of the heated parts of the girder. The failures which were fairly numerous were all identical, being fractures of the vertical butt welds between the new web streng-

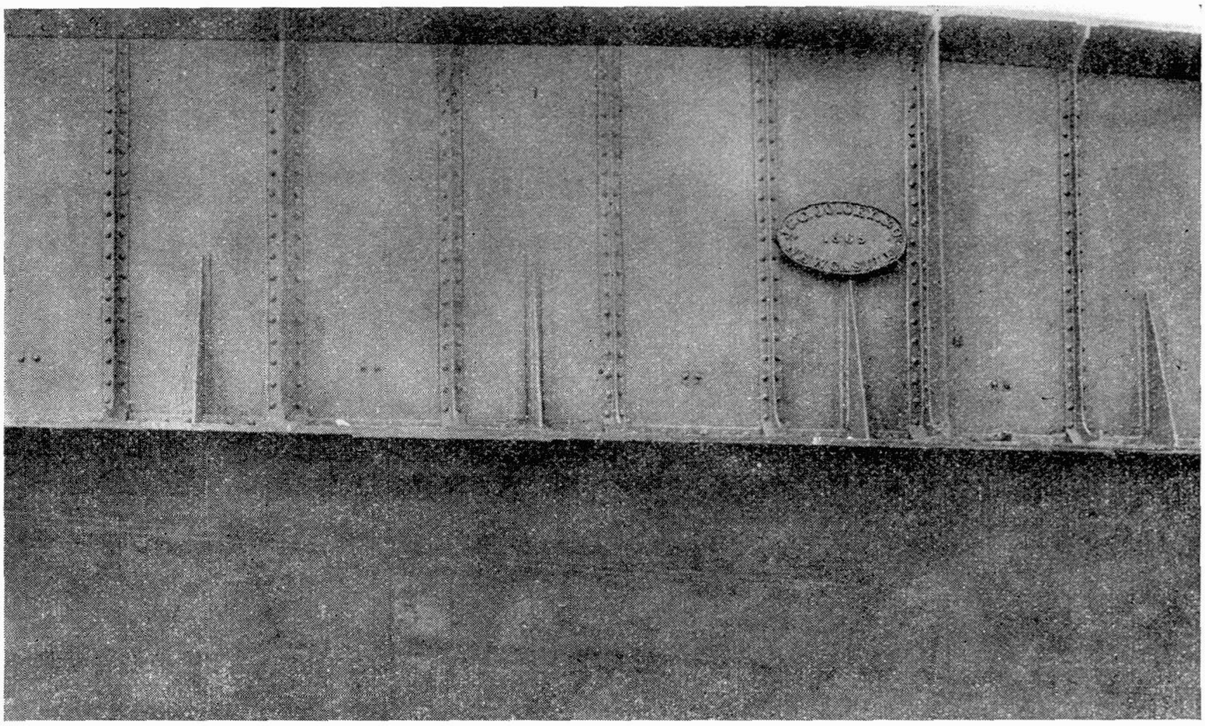

Fig. 3.
Lichtbild des reparierten Stegbleches. Außen- und Innenansicht.
Ame réparée. Vue extérieure et intérieure.
Photograph of repaired web; outside and inside.

thening plate and the toes of the "*T*" and angle stiffeners. Fig. 4 shows details of construction.

As will be seen from the section it was intended that the welds should not only unite the strengthening plates and the toes of the "*T*" or angle stiffeners, but also the old web itself. On cutting out the fractured welds, as shown in Fig. 5, it was found, that good penetration had been obtained between the strengthening plate and the "*T*" but that there was no penetration in the case of the web, as will be seen from the photograph of a part of the "*T*", strengthening plate and web which were cut out in order that the cause of the fracturing might be ascertained.

The gap between the strengthening plate and the "*T*" had been prescribed to be 1/16″, but apparently this had not been worked to in all cases, the weld cut out was barely 1/16″ and it appeared that the strengthening plate had not been in full contact with the web plate. Out of 48 welds 17 were found to be fractured, these were all cut out and in not a single instance had penetration with the web been secured. The practice is now to make the gap 1/16″ min. the face of the chamfer the same, using a No. 10 electrode with a slightly higher

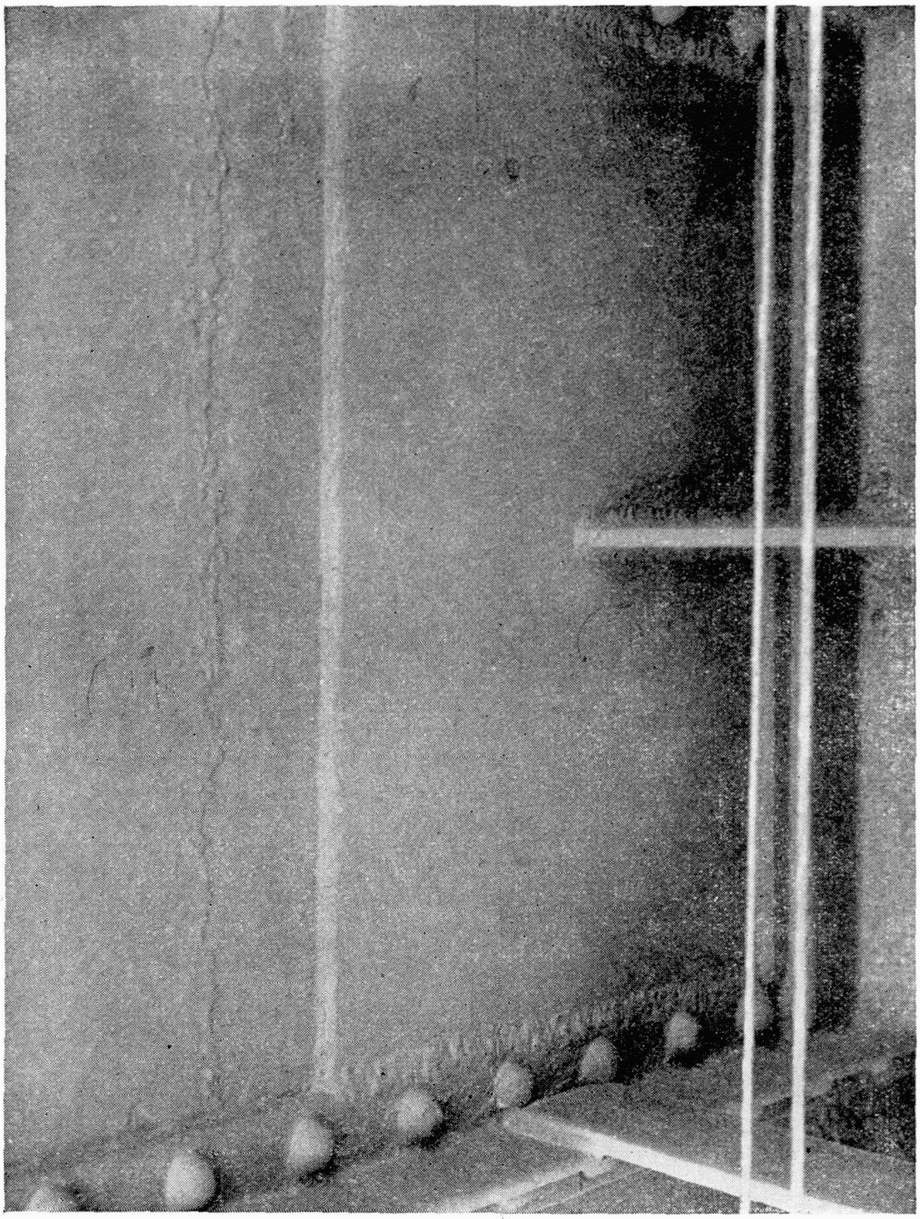

Fig. 4.
Einzelheiten von Stegblechreparaturen. (Brücke No. 2, Monkwearmouth Güter-Linie).
Details de réparations de l'âme. (Pont n° 2, ligne Monkwearmouth — trains de marchandises).
Details of repairs to web. (Bridge N° 2, Monkwearmouth Goods Branch).

amperage than normal, which in the case of so light a run is not likely to overheat the mild steel. Fig. 6 shows the repaired girder with removed section (Fig. 5) which was not replaced, the hole was merely fillet welded round the edges as there was ample section left for strength. The bottom flange, which the photo shows covered with gunite, was badly wasted by blast from engines, had new flange plates welded on, but as this work was perfectly successful it does not concern us at present.

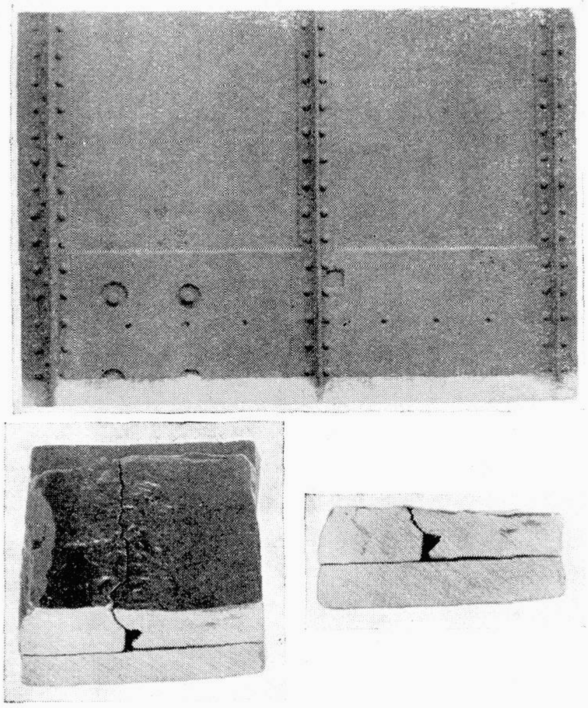

Fig. 5.

Schnitt durch die gerissene Schweißung.
Coupe à travers la soudure rompue.
Section of fractured weld.

Another failure of similar nature was the fracturing of the butt weld between two channel way beams, of construction as shown in Fig. 7.

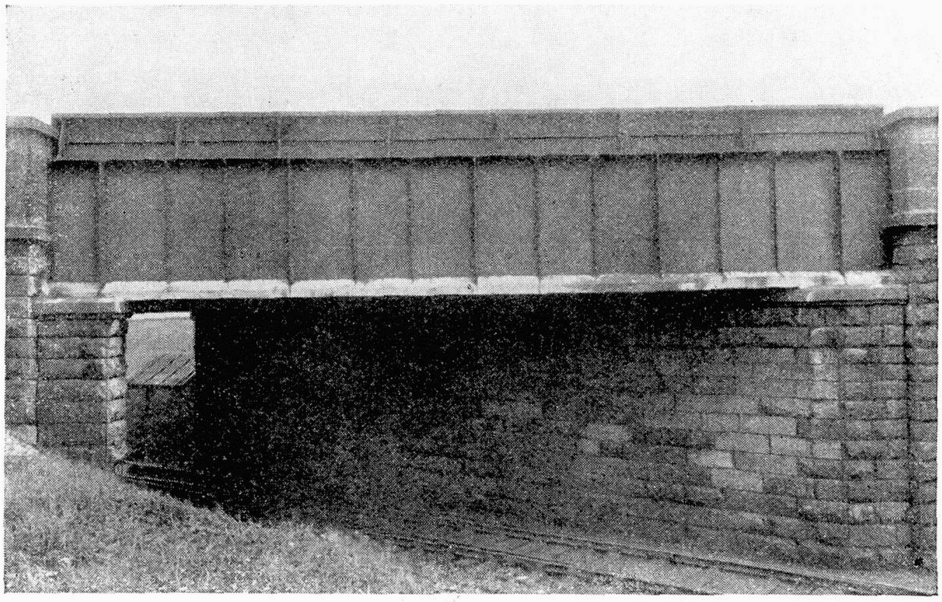

Fig. 6.
Träger mit repariertem Stegblech. — Poutres dont l'âme est réparée.
Girder showing repaired web.

The gap intended to be 1/16″ wide was somewhat wider when fitted and to expedite the work two welders were put on to make the weld. One of the men had advanced further than the other when they left off for a meal, when shortly afterwards the weld fractured with a loud report. The weld after all the old weld metal had been removed and the butt prepared afresh was rewelded by one man commencing at the centre, working slowly without unduly heating the channels, as it was thought that the unequal and rapid cooling off of the overheated weld had been the cause of the fracture. As this, as well as all the other welds of this construction, all have remained perfect, it is clear, that butt welds of free ended sections must not be allowed to become too hot when being run.

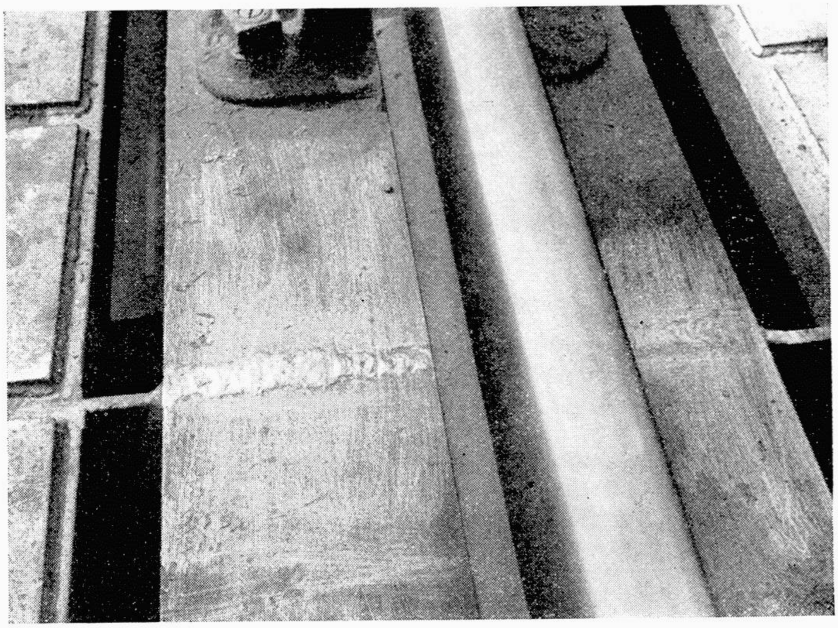

Fig. 7.
Stumpfschweißung an Längsträger (Schienenträger). (Brücke No. 41, Denaby Linie).
Soudure bout à bout des longerons (sous les rails). (Pont n° 41 de la ligne Denaby).
Butt weld of way beam. (Bridge No. 41 Denaby Branch).

Shock.

What may be termed shock fractures of welds are fractures which have taken place unexpectedly not long after the welds have been made, and are caused by the sudden application of excessive loads. In a sense they are fatigue fractures, but the latter take place after a considerable number of repeated strains. The strains may fluctuate between alternate compressive and tensile maxima or from nil to a compressive or tensile maximum back to nil or even from a pre-load, compressive or tensile, to a maximum of the same category back to the original pre-load, the impulses occurring in very rapid succession.

Shock fractures in connection with completed works confine themselves to a few fractures on one bridge.

The welded work in question comprised the connections of a welded way beam, illustrated in Fig. 7, further details are shown in Fig. 8.

As will be seen the rails and chairs were bolted to the webs of the steel channels, which again were welded to rolled steel beams resting on the top flanges of the old rail bearers to which they in turn were welded. At each cross girder gusset plates were welded to the vertical channel flanges and to the top flange plates of the cross girders, for the purpose of taking up any side sway of the steel way beams (consisting of the rolled steel beam and the channel). Two of these gussets at one end of the bridge, where there is a switch, have fractured. The position of the fractures are indicated on Fig. 7 and on the photograph of the bridge end, Fig. 9.

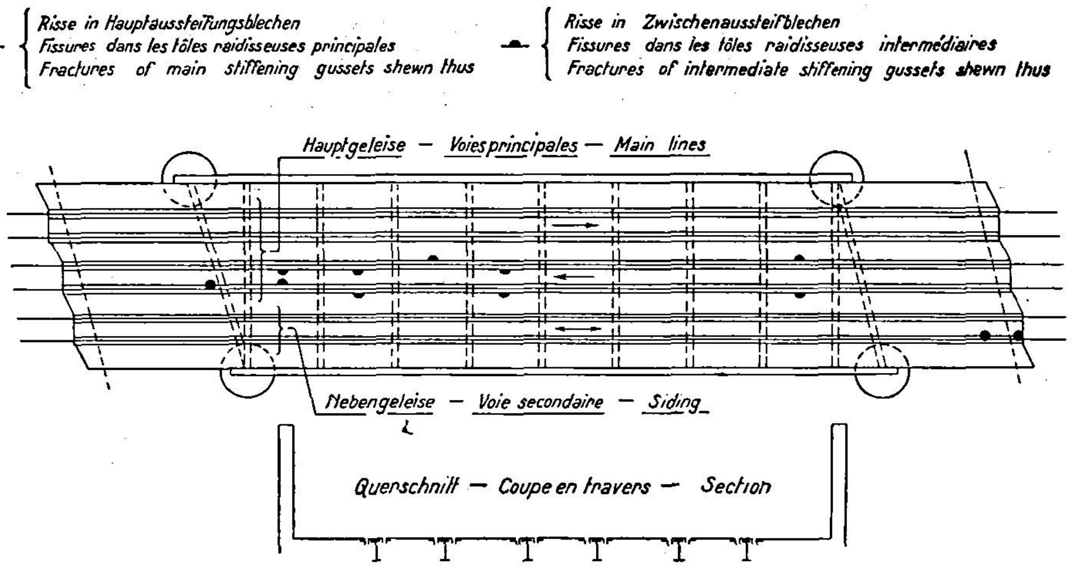

Fig. 8 a

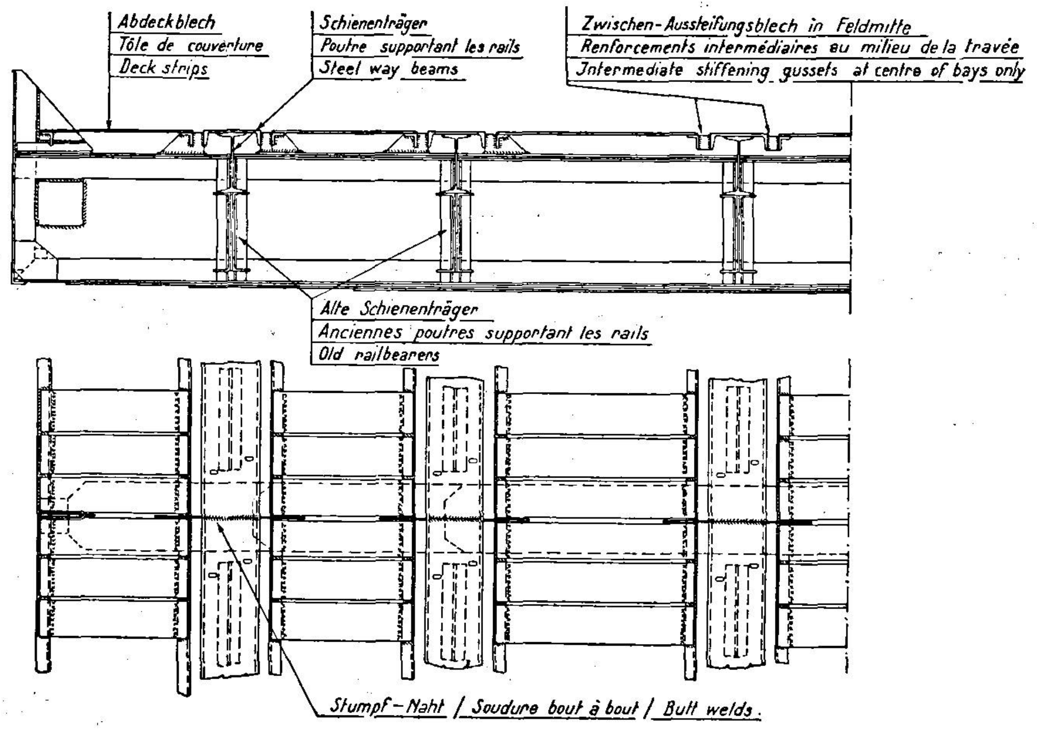

Fig. 8 b.

Grundriß und Einzelheiten der Brücke. — Plan et détails du pont.
Plan and details of bridge.

As will be noted the position of the fractured gussets occur on the convex side of the curve, where tension will be produced through the rigidity of the wheel base of engine counteracting the centrifugal action. A close up photograph of the fracture is shown in Fig. 10.

Similar fractures have also developed where some small tie plates have been welded on to the way beam channels, for the purpose of adding support to the angles carrying the deck strips, the ties being welded on between the deck angles and the channel flanges along four of the channels (see Fig. 8). These ties also take up some side sway of the way beams. Of 348 welds (two to each tie) 10 have fractured, probably due to shock, as they fractured shortly after the centre track, where they were located, was brought into use. On close examination of the fractures it has been found that the welds are all indifferently

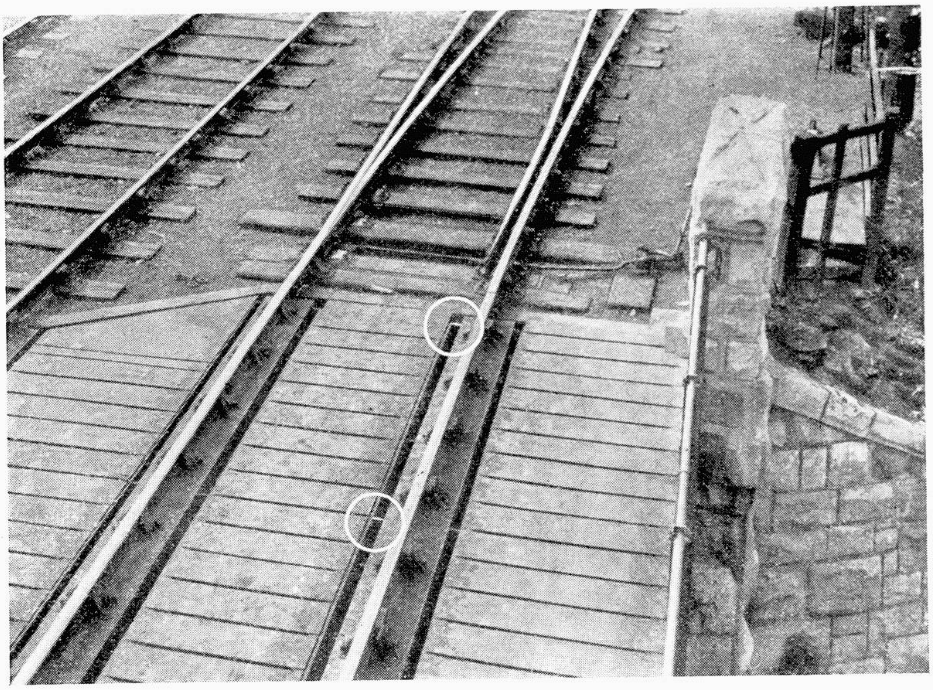

Fig. 9.

Brückenende und Weiche. Lage der Risse.
Extrémité du pont et aiguille. Emplacement des fissures.
Bridge end and switch. Position of fractures.

executed; unfortunately the conditions under which the work was carried out were adverse, as it was necessary to complete within limited time under traffic pressure, and during wet and cold weather. It is therefore possible, that the fractures are initially due to internal welding stresses. After more than a year no further fractures have taken place and this to a certain extent indicates that this may be the case. It is also possible that the construction is too rigid and that it is necessary to have a certain amount of flexure of the way beam to withstand the sway of the engines when travelling.

Fatigue.

The usual nature of fatigue fractures have been described above under shock. So far only one case has come under observation. This is a welded double line trough girder bridge carrying four interlaced main lines.

The bridge shown in Fig. 11 has a steel deck which is stiffened with $2'' \times 3/8''$ flats on edge. Besides these there are transverse diaphragms between the two sets of troughs. When erected it was found that the brick abutments when bared were not very good, showing fractures below the bedstones on which the troughs would be seated. The bedstones were carefully levelled before the girders were erected on them, but possibly owing to the more rigid construction and absence of cushioning due to steel bearers instead of timber way beams in the troughs, the fractures in the abutments increased and caused unequal depression of the girder ends which set up transverse bending moments

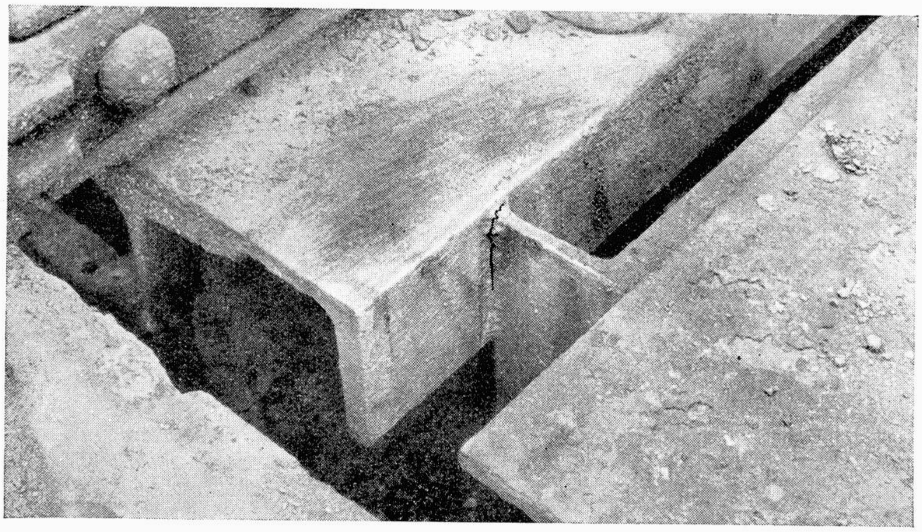

Fig. 10.
Gerissenes Knotenblech. — Tôle fissurée d'un gousset. — Fractured gusset.

in the deck stiffeners, which were not anticipated. Such transverse bending moments would be additional to the bending moments caused by deflection of one set of troughs when one trough was loaded and the other was not simultaneously loaded by passing traffic.

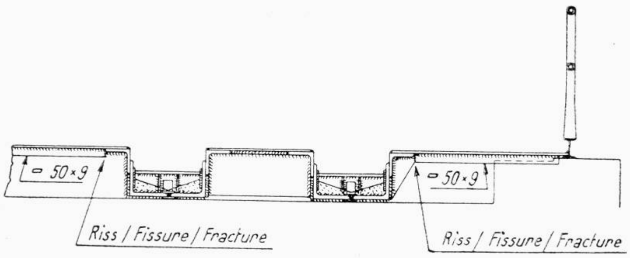

Fig. 11.
Halber Schnitt durch die Brücke. (Brücke No. 25, York und Doncaster).
Demi-coupe à travers le pont. (Pont n° 25, York et Doncaster).
Half section through bridge. (Bridge N° 25, York and Doncaster).

As mentioned above the bridge carries two separate main lines, and is therefore very busy, the average number of trains per 24 hours being 250 representing about $5\frac{1}{2}$ million stress variations annually, the traffic being about the same in both directions. Under these circumstances it will be realised that the enormous frequency of stress reversals which occur will tend to produce fatigue effects in parts unduly stressed. From the construction it will be appreciated that severe stresses must be set up in the transverse stiffeners of the deck plates duo to such unequal deflections, which are enhanced by the settlement of the bedstones.

The welds connecting the deck plates and the $2'' \times 3/8''$ stiffeners are not very large and even normally they are rather heavily stressed, with the consequence that 19 out of 20 have fractured.

On the other hand none of the welds of the diaphragms in the four foot have fractured, as there is very little variation in the deflections except for the differences in settlement due to the bedstones. Recognising the importance of more rigid supports required in the case of welded structures, especially girders,

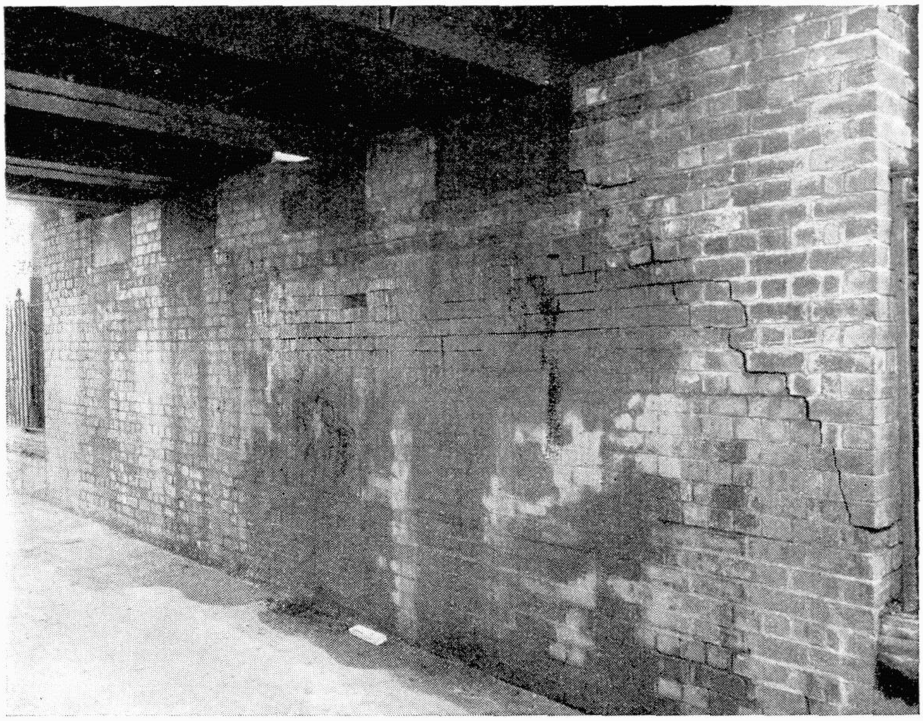

Fig. 12.
Gerissenes Widerlager. — Culée fissurée. — Fractured abutment.

heavy steel grillages embedded in concrete are now always provided. These grillages are most carefully and truly set, so as to reduce unequal settlement of adjoining troughs to a minimum. In addition to the grillages the stiffeners for the deck plates are not carried through and welded to the flanges, thereby permitting the deck plates to flexure and so give a little play which absorbs unequal deflections.

In addition to the instances described failures have been reported in connection with the repairs by electric welding of the spandril frame above the solid plate arch ribs of a large wrought iron arch viaduct of several spans. Here 4 butt welds out of 109 fractured through the centre of the weld on the line of the original fracture, due to overstress.

The only other fractured welding observed are three short overhead fillet welds on one of the first bridge strengthenings carried out, a plate decked trough girder. The cause of the failures, apart from the awkward situation of the weld on the back of the deck angles, was because no penetration of the weld metal had been effected owing to the presence of water between the angle and the deck plate.

The general conclusion as to the safety of welds so far arrived at by the writer is, that it is not necessarily a low working stress, which will determine whether fractures are formed or not, but the toughness and the ductility of the deposited weld metal and the alloy formed by it and the parent metal at the junction of weld and work. Naturally penetration is a sine qua non. It is therefore seen that there are two factors which determine the reliability of welding before anything else, the electrodes and the efficiency of the welders.

The total length of weld failures which have taken place in welding work carried out by Mr. Miller with his own staff is 44 lin. feet, out of a total footage of about 136 900 lin. feet representing 0,032 %. Of the comparatively small amount of welding let by contract, so far no weld fractures have been reported. All welding, whether domestic or contract, has been carefully inspected visually when completed and the slag has been removed by chipping. If the condition

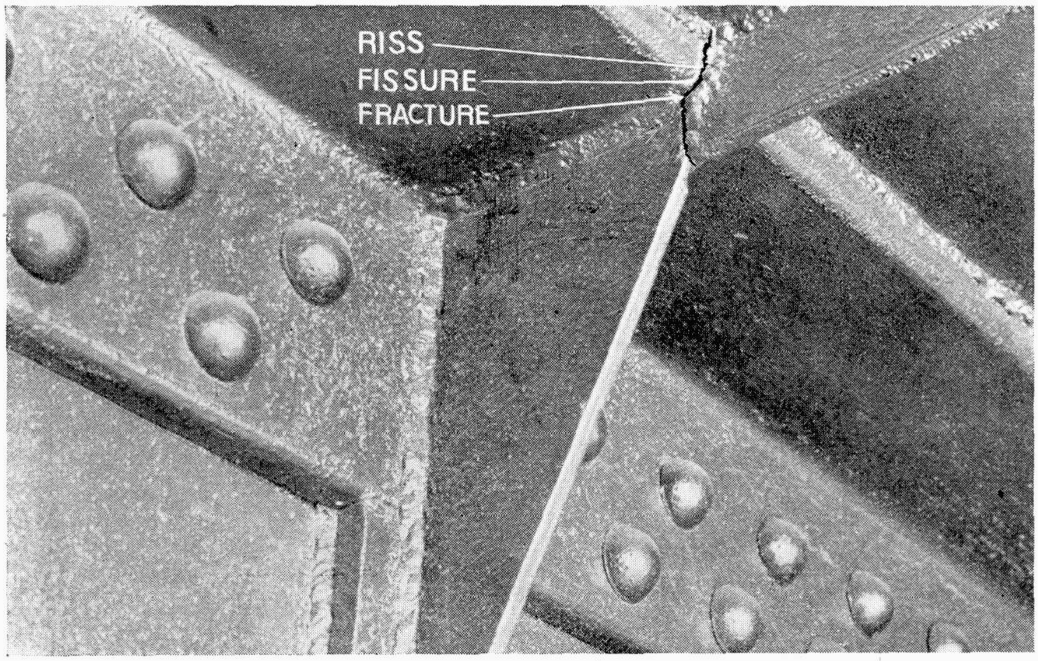

Fig. 13.
Gerissene Schweißung. — Soudure fissurée. — Fractured weld.

of the weld seemed to indicate faulty work, the weld has been chiseled out and rewelded. Very little welding has, however, been found not to be up to the requirements, amounting all together to a few hundred lineal feet only. No other method of testing by X rays or other means have been made use of.

Summary.

The common feature of the recorded failures, except in the case of the failure through fatigue, is, that all the fractures took place shortly after completion of the welds as they were cooling down or shortly after the application of the live load. Further that after the fractures were re-welded so as to secure adequate fusion, a repetition of the fracturing of such repaired welds have not taken place. From these two facts one may conclude, that provided the work is carried out consciensiously with high class covered electrodes such as were used in this case, no further failures need to be feared.

In the case of the welded connections which failed through fatigue the welds in question were obviously too light to stand the repeated strain, and

that instead of aiming at obtaining a rigid connection a more flexible one is to be recommended, as such flexibility will permit the parts suffering varying degrees of deflection to adjust themselves. Furthermore it is to be recommended that in the case of composite structures, like a two track bridge, the whole structure would be seated on firm supports, for preference such as continuous steel grillages will provide.

Zusammenfassung.

Die beobachteten Risse haben mit Ausnahme von Ermüdungsbruch das gemeinsam, daß sie alle unmittelbar nach beendigter Schweißung während des Abkühlens oder kurz nach Einwirkung der Verkehrslast entstanden. Eine Wiederholung der Risse fand nicht statt nach erfolgter Neuschweißung, wenn auf gute Schmelzwirkung geachtet wurde. Aus diesen zwei Tatsachen geht hervor, daß das Auftreten von Rissen nicht zu fürchten ist, wenn die Arbeit gewissenhaft mit umhüllten Elektroden, wie sie hier zur Verwendung kamen, ausgeführt wird.

Im Falle von Ermüdungsbruch scheint es, daß die Schweißnähte offensichtlich zu leicht waren, um die Spannungen zu ertragen. Es ist daher besser, das Ziel in biegsamen Schweißnähten zu suchen als in starren. Bei besserer Biegsamkeit können sich diejenigen Teile, die verschiedenen Graden von Durchbiegung ausgesetzt sind, besser anpassen. Außerdem ist es sehr zu empfehlen, daß bei Konstruktionen verschiedener Zusammensetzung, wie im Falle der zwei beschriebenen Eisenbahnbrücken, das ganze Tragwerk auf solide Fundamente zu ruhen kommt, vorzugsweise durchgehende Trägerroste.

Résumé.

Les fissures observées — excepté le cas de rupture par fatigue — ont toutes ceci de commun qu'elles se sont amorcées immédiatement après l'achèvement de la soudure, pendant le refroidissement, ou alors peu après l'effet des surcharges mobiles. Les fissures ne se sont pas reproduites après le renouvellement des soudures, tant que l'on a cherché à obtenir une bonne fusion. De ces deux faits, il résulte que l'on n'a pas à craindre de fissures lorsque le travail est fait consciencieusement et avec des électrodes enrobées telles qu'elles ont été utilisées ici.

Dans le cas de rupture par fatigue, il est certain que les cordons de soudure, trop faibles, n'ont pu suffire aux contraintes. Il est donc préférable de rechercher des cordons de soudure facilement déformables, plutôt que rigides. Lorsque la déformabilité est meilleure, les parties soumises à des fléchissements différents peuvent mieux s'adapter. En outre, il est très recommandable de donner des bases solides à des constructions qui se composent d'éléments divers, comme c'est le cas des deux ponts-rails décrits; on choisira de préférence des grillages continus d'acier.

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