Experience obtained with structures executed in Germany

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Experience obtained with Structures Executed in Germany.

Erfahrungen bei ausgeführten Bauwerken in Deutschland.

Observations sur les ouvrages exécutés en Allemagne.

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I. Introduction.

It was on the initiative of the Expert Committee for Welding Technique of the Verband Deutscher Ingenieure (German Association of Engineers) that welding was first applied to steel structures. In 1930 that Association published its "Instructions for the Application of Welding to Structural Engineering".[1] About the same time information was received from America that welding had already been used over there in the construction of several bridges, and a draft of the American "Instructions for Welded Bridges" was published by G. D. Fish in the Eng. New. Rec. of the 22nd August 1929.[2] The result was that great interest was aroused in Germany with regard to the welding of steel structures, although the publications did not give entire satisfaction as it appeared that the welding had been carried out in a somewhat desultory manner and further that formulas for computing the welded joints were available only for uniaxial stresses. Welding was not carried out on a large scale in Germany until further publications[3] showed how the resistance of welded joints could be determined when subjected to bending stresses and to duoaxial tension. These suggestions, together with results of experiments carried out on welded metal structures (the test pieces being subjected mainly to static stresses) led in 1931 to the framing of German official "Regulations for Welded Steel Structures (Structural Engineering and Bridges)" which then came into force for the whole of Germany. The publication of these Regulations was followed by intensive development of welding in Germany. At that time it was commonly held that the structural parts of welded bridges could be calculated in the same way as those of riveted bridges, the only difference being that the thickness of the welded joint was altered according

[1] *Die Elektroschweißung* (Electric Welding) Part I., 1930.

[2] *Dr. R. Bernhard* (Engineer): *Die Elektroschweißung*, Part 2, 1930, and *Bautechnik* (Building Technique), 1930, Page 117.

[3] *Dr. Kommerell* (Engineer): „Computations, Details of Structure and Welding of Railway Bridges" Published by Messrs Wilhelm Ernst & Son, Berlin, 1930.

to the surge load and alternations of the stresses. However, even at that time all were agreed that enlightenment could be obtained only by carrying out systematic fatigue tests. The experiments which were carried out, and which necessitated an outlay of some 50,000 Reichsmark, have been fully dealt with in the Report which I submitted to the Berlin Congress under the title of "The Influence of Frequently Alternating Loads on Welded Structures". In that paper I further expounded the bases of computation according to the knowledge obtained from the experiments.

In the present paper I propose to giving examples taken from actual practice and experience gained in connection with the welding of bridges. Wherever it appears expedient, special reference will be made to building methods which are now out of date and which have therefore been discarded in Germany. Briefly, the advantages of welding applied to building as compared with riveting are as follows:

1) The drilling of rivet holes and the hammering of rivets is almost entirely eliminated.

2) There is no weakening of the cross-section by rivet holes, and this advantage, added to the fact that the welding does not require angle sections, results in a reduction of about 15 to 20 per cent. in weight.

3) Angles and corners of framework, often complicated by riveting, can be carried out far more simply with welding.

4) The appearance of welded structures is more attractive and satisfactory than that of riveted ones.

5) It is far easier to vary the thickness of plates when welding than when riveting (for instance, thickness of the web).

6) When using plate girders and applying welding, it is easier to increase the span than when using riveting for joining the various parts. In Germany 30 metres used to be the approximate limit for the span of riveted girders, while welded plate girders have been made with a span of up to 54 metres, so that at present there is no need for us to weld lattice bridges. The difficulties which arise when erecting welded lattice bridges in consequence of the contraction of the welded joints and the reduced resistance of the latter to eccentric application of force, together with a weakening in the zones where the lateral fillet welds and the parallel fillet welds start, have led us to desist, at any rate for the present, from constructing lattice bridges which are entirely welded. Meanwhile, there is no reason why later on, when constructing large bridges, the booms and the various web members should not be welded in the workshop, and the connections between the web members and the gusset plates riveted on the building site. Neither is there any reason why riveting should not be applied when welding plate girder bridges, in cases where riveting seems more advantageous than welding — for instance, at the joining of the cross beams to the main girders, for the wind bracing, etc. As the stresses are mainly static in structural engineering, welding can be applied unhesitatingly to lattice work.

The structural details of welded girder bridges have been very considerably influenced by the regulations determining dimensions and in particular by those laying down the permissible stresses.

Originally fillet welds were considered better and more reliable than butt-welds, but now opinion on this point has veered round entirely as a result of endurance fatigue tests carried out with oscillators. The reason for this lies in the fact that in the case of butt welds the flow of stresses is a natural one, while for fillet welds the stresses have to be deflected and this sets up peak tensions with consequent notching effects. It is common knowledge that welded joints do not resist these notching effects satisfactorily.

II. Welded Plate Girders.

A. Webs.

Welding offers an advantage when using plate girders in that it is possible to vary the depth and thickness of the web more easily, and thus, even with an increased span between the supports, girders with continuous boom plates

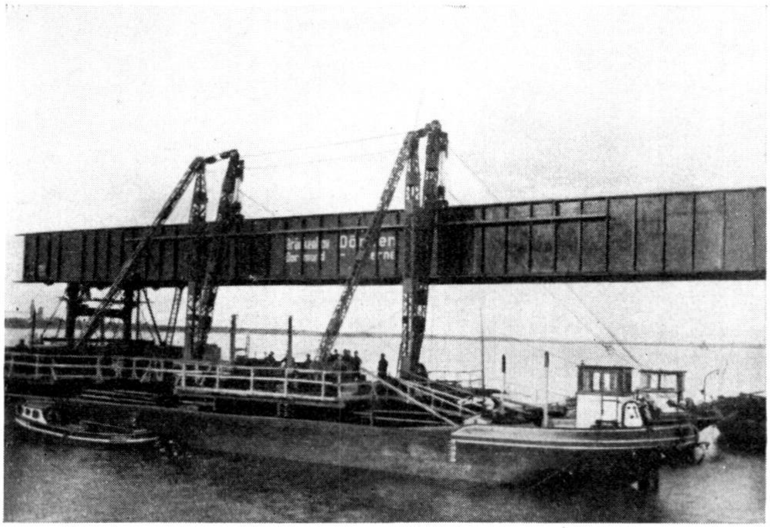


Fig. 1.
Transporting welded plate girders of 54 m length.

running right along the whole length (and in certain circumstances girders with curved booms) can be used. This is satisfactory in that joints in the booms can be dispensed with, thus improving the appearance of these structures and further allowing the main girders to be made and welded entirely in the workshop. The German State Railways have transported main girders with a span of 54 metres by rail over long distances in order to take them from the workshop to the building site; they were loaded on board ship by means of cranes, brought to the building site and then deposited on the masonry there. (Fig. 1.)

In Germany parallel girders with boom plates running along the whole length are often used in order to obtain a satisfactory and restful appearance. This type of girder is preferred in spite of the fact that in certain parts the cross section cannot be used to full advantage; on the other hand, the elimination of flange plate joints and flange plates of varying thicknesses lowers costs and simplifies construction. The appearance of structures of this kind is far more satisfactory than that of structures with flange plates which are stepped in thickness. Moreover, it is only quite recently that recourse has been had to the possibility of varying the thickness of the web according to requirements. This need arises frequently with continuous girders when intense bending stresses

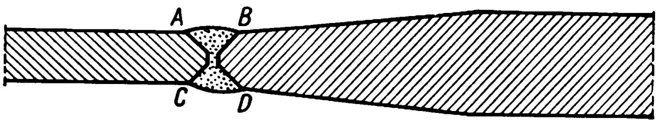


Fig. 2.

occur above the supports. With thicker webs of this kind the tension in the neighbourhood of the boom joints can be reduced as desired. Fig. 2 shows how the passage from one web thickness to another is effected by means of double-bevelled joints. Meanwhile it would be advisable to reduce the thickness

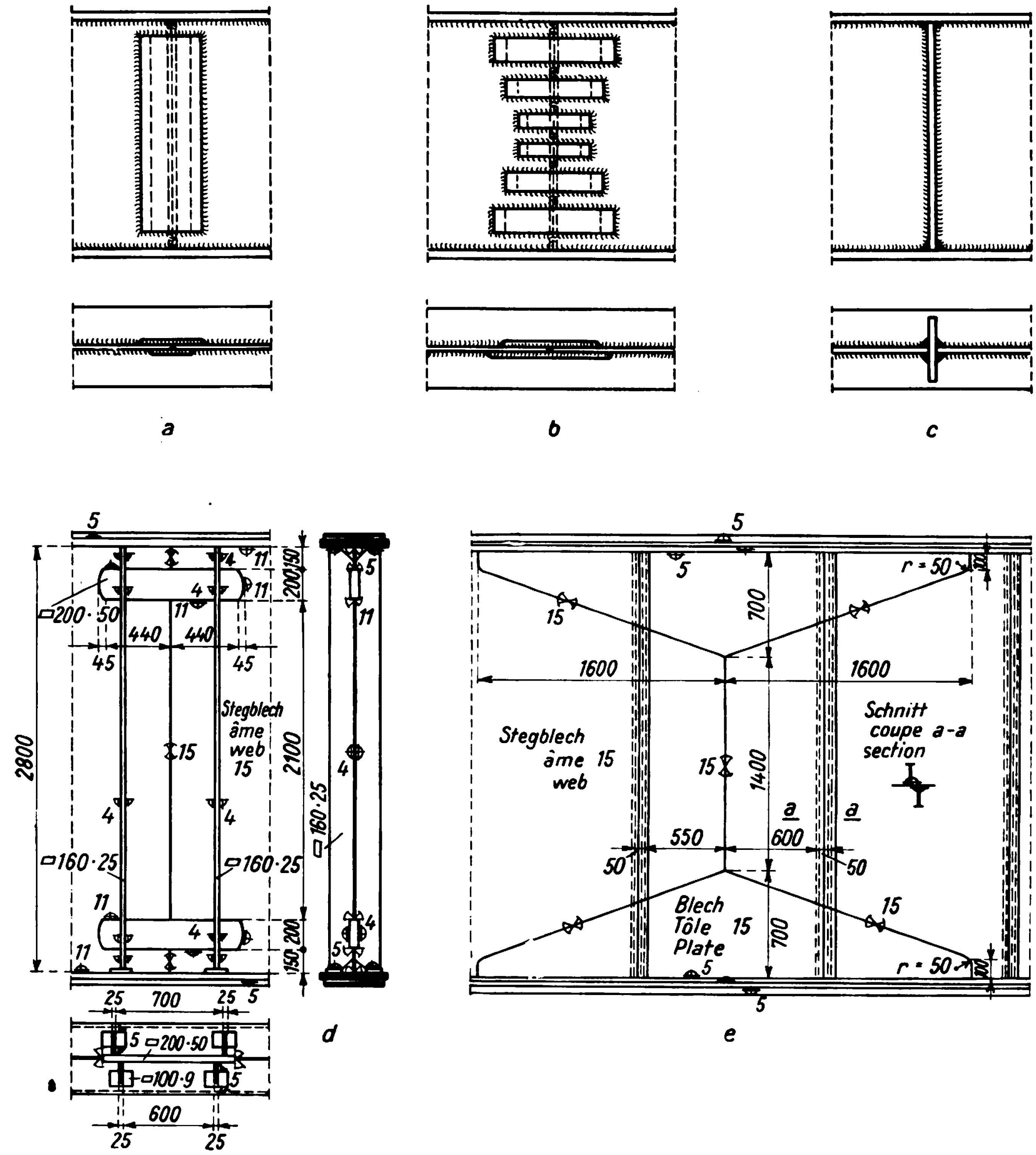


Fig. 3a—e.
Obsolete types of welded web jointing.

of the heavier plate near the joint in order to obtain a gradual transition from weld to parent metal by grinding down the metal at points A, B, C, and D. The introduction of butt-welding for web jointing at any given point was a result of the new Regulations governing calculations, and this method is now used exclusively. Before these Regulations came into force, all kinds of building methods, now out-of-date[4], were applied (for examples see Fig. 3, a to e) because formerly the permissible maximum stress for butt welds was only 0.75 σ perm.

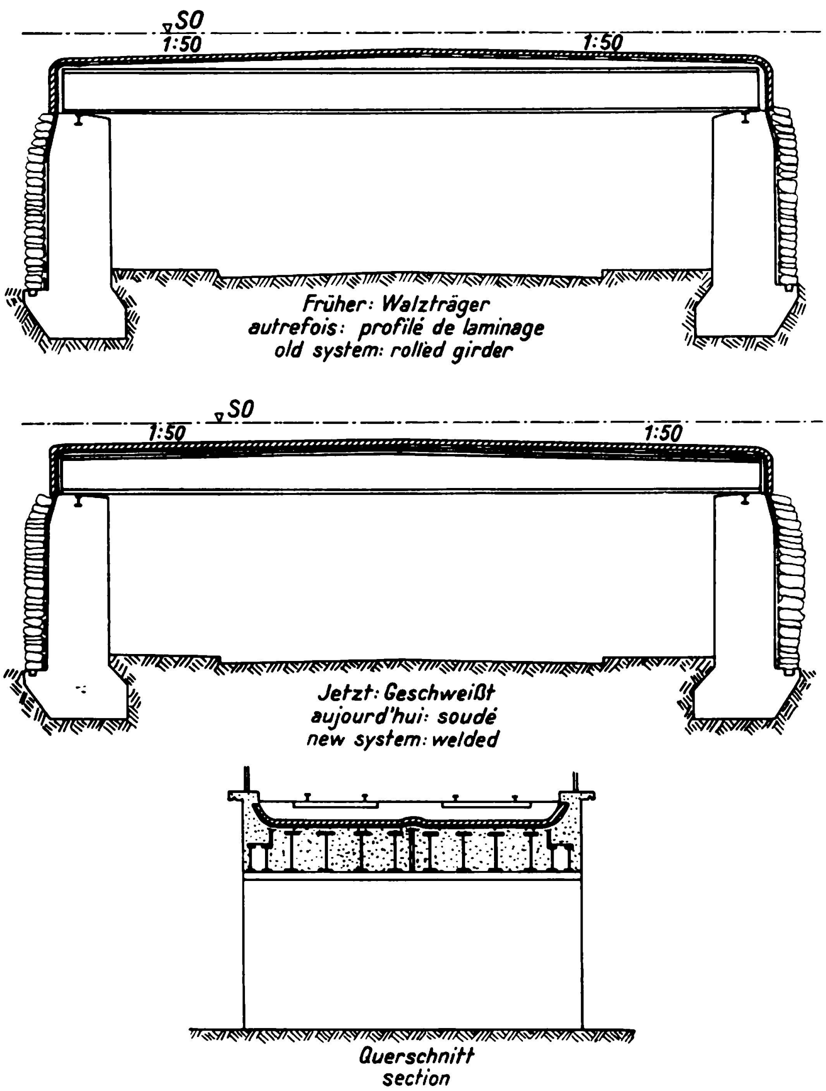


Fig. 4.

There was something artificial about these structural methods, it was difficult to apply them on account of the shrinkage stresses which were set up, and our present knowledge of bridge construction does not supply very definite data concerning their behaviour under continuous stress.

R o l l e d G i r d e r s e n c a s e d i n C o n c r e t e. These are very extensively used, being in great favour, not only for railway bridges but also for ordinary traffic bridges. The advantages of this type of construction lie in simplicity

[4] See *Schaper: Feste stählerne Brücken* (Rigid Steel Bridges) Figs. 92—97. Published by Wilhelm Ernst, Berlin, 1934.

of execution; the work can be carried out by ordinary contracting firms, costs of upkeep are not heavy, uniformity of the superstructure is secured because the ballasting above the structure can be carried out in a continuous manner. As draining requires an inclined concrete surface, the height of the structure had to correspond to the height of the girder at the lowest point of the concrete surface. Where rolled girders are used, the concrete covering must, of course, be unnecessarily thick at the highest point of the covering, so that, in the case of railway bridges, the span was limited to approximately 15 metres. When using welded girders deeper in the middle than at the ends (Fig. 4), the concrete covering can have a uniform depth; fuller advantage can thus be taken of the girders because they adapt themselves better to the bending moment curve and this means that girders of wider span may offer economic advantages when welding is used.

B. Booms. (Flanges.)

In the earliest examples of welded structures, the various members consisting of angles and plates were merely joined by fillet welding[5] (See Fig. 4, a and b),

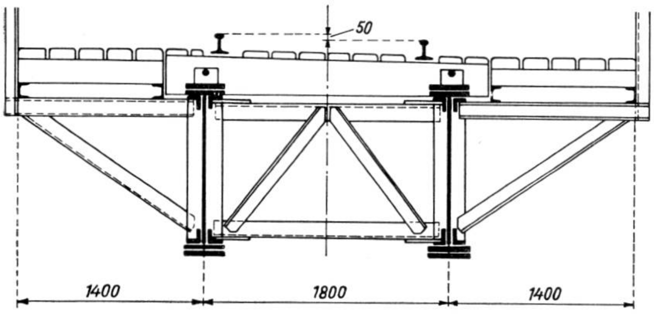


Fig. 4 a.

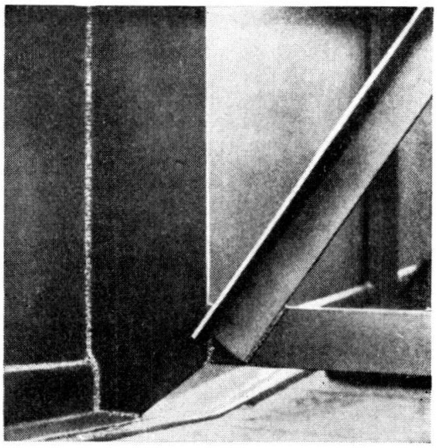


Fig. 4 b.

instead of with rivets. However, it soon became apparent that quite new shapes would have to be evolved and that angle irons for connecting the various parts of the structure could be dispensed with. Before long, broad flange girders[6] were cut up and used as booms for plate girder, the webs being joined by butt-welding to inserted plates whose depth corresponded to the bending-moment curve. Another development that followed soon after was the introduction of special rolled sections as booms, and these were designed with a view to placing the weld which joined the web to the boom in those parts of the structure which were less subjected to stress. (See Fig. 5, a, b and c).

In order to join the broad flat plates, forming booms to the web, the original practice was to employ a series of fillet welds, as according to calculation these

[5] See *Bondy:* „Ausgewählte Schweißkonstruktionen" (Examples of Welded Structures), Vol. 1, Pp. 79/80, Published by the V. D. I. (Association of German Engineers) Berlin, 1930.

[6] *Ulbricht:* „Stahl und Eisen", (Steel and Iron) 1931, P. 253, „Bautechnik" (Building Technique) 1931, Pp. 263, 332, 497/8.

seemed to meet the shearing stresses satisfactorily. However, it soon appeared from experiments[7] carried out with girders that resistance to fatigue was greater in the case of girders joined with solid fillet welds than with lines of interrupted fillet welds (starting and ending of fillet welds). This explains why the new German Regulations for plate girder railway bridges prohibit the use of interrupted welds and slotted welded joints for bridges. (Structural engineering does not require a prescription of this kind.) In order to avoid the web being too thin as compared to the flanges, the new regulations demand the calculation of the maximum shearing stress in the neutral axis and the main stress of the web in the transition zone between web and flange. Owing to the preponderance of the influence of the bending stress, the main stress increases proportionately to the increase of distance between the welds which connect boom and web and the neutral axis, and this explains why the special sections shown in Fig. 5 should

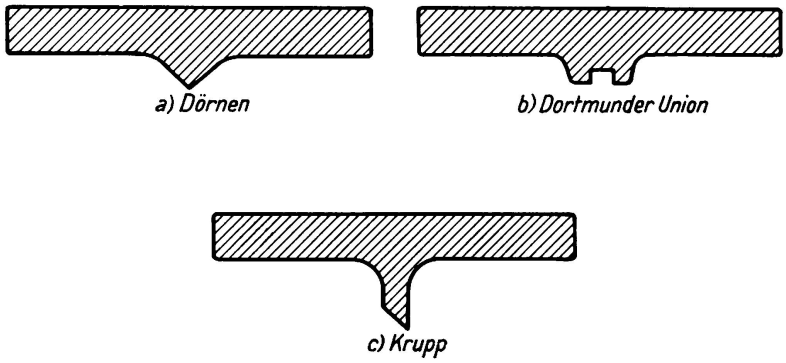


Fig. 5.
Special sections for flanges.

be used in preference to the ordinary broad flats with fillet weld connections. Given the same carrying capacity, the thickness of the fillets should be approximately:

$$a = \frac{t}{2}$$

(See Fig. 6.) Less weld metal is required with the special sections shown in Fig. 5, a, b and c (there is less shrinkage stress). The flow of forces is more favourable. Better execution is secured with butt-welding and they can be more easily X-rayed.

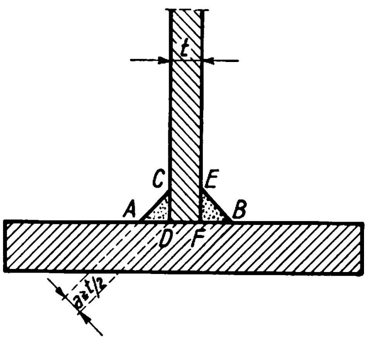


Fig. 6.
Obsolete.

[7] *Hochheim:* „Mitteilungen aus den Forschungsanstalten der Gutehoffnungshütte", (Communications received from the Research Bureaux of the *Gutehoffnung* Foundries, 1932, 1, P. 225. (See Kommerell, Commentaries, Part II, p. 65.)

When making fillet welds there is always danger of the weld metal not filling the root properly, in other words, inroad at AB of the boom or flange and at CD and EF may be defective or inadequate.

When joining the web to thick heavy flange-plates (in Germany such work has been carried out on thicknesses up to about 8 cm), it should be remembered that the shrinkage stresses with these heavy plates will probably be very great, so that care must be taken to provide for the parent metal and the weld metal being in a position to attenuate these high shrinkage stresses by plastic deformation as soon as the shrinkage stresses, together with the first loading, reach the yield limit. The parent metal, when cooling rapidly, should be of such nature as not to show any signs of hardening. This is particularly important when dealing with alloyed steels (St. 52). For the same reason the finished weld must be ductile, not brittle.

Economic considerations require that when constructing large bridges the boom cross-section be adapted to the bending moment-diagram. The earlier Regulations prohibited the transmission of tensile stresses by butt welds alone, and the only remedy to this lay in adding further flange plates by means of fillet-welding. Even to-day this method cannot be entirely avoided, in spite of the fact that butt-welding, allowed under the new Regulations, offers the possibility of a smooth transition from a thin to a heavier plate. The transition, however, must be gradual, because sudden changes in cross-section react unfavourably to fatigue resistance. Although the new Regulations allow uncovered butt-welds for flanges under tensile stress, a precautionary measure is at present applied by which these joints are strengthened by either riveted or bolted supporting straps which are capable of taking up the whole tensile stress in the flange, should the butt weld break.

I believe this precautionary measure is merely provisional and I feel sure that later on, when we have more knowledge at our disposal and the absolute soundness of these butt welds has been proved by repeated X-ray tests and by vibrating the structures by means of mechanical oscillation, butt welds will be used for flanges subjected to tensile stresses without the addition of these safety straps. This will first be done with bridges of St. 37, and in this connection it must be remembered that the Regulations require that these butt welds, having an angle of less than 45° (Fig. 7) must be so welded that the root is

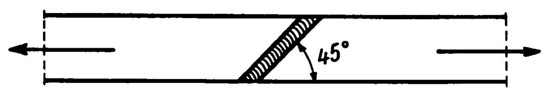


Fig. 7.

perfect, and further they must be tested by X-ray methode. (We would recommend that the very first pass of weld-metal should be X-ray tested to make sure there are no cracks.) Further, the weld should be carefully machined or ground so that the transition from weld to plate is even and gradual and that a smooth surface is secured. With such joints it is quite easy to obtain a surge-load strength of 22 kg. per sq. mm, while the Regulations allow a tension of only $0.8 \times 14 = 11.2$ kg per square mm, that is to say, approximately half of what is actually obtained.

As the reduction of the permissible stress to eight-tenths (0.8) refers only to the tension flange, the result is a very desirable one, namely, the automatic displacement of the flange-plate joint in the tension and compression flanges. Compression flange plates which are not directly (continuously) joined to the web must, if their width is more than thirty times their thickness, be connected by rivets or bolts in order to prevent warping. (Riveting fully answers the purpose even for heavy plates and is cheaper than using bolts.) Continuous fillet-welds, when properly executed, offer the same surge load strength as do butt welds. Meanwhile the permissible stress must be reduced to comply with the Regulations concerning all end fillet welds and all points where side fillet welds start or end. This is a point which must be borne in mind, not only with reference to the flange plates which are joined by fillet welds, but also when assembling other structural parts, such as, cross beams, wind bracing gusset plates, etc.

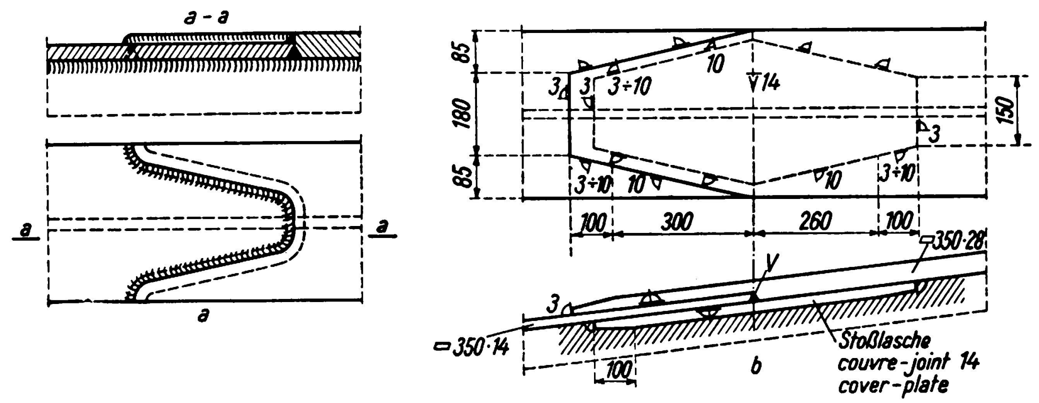


Fig. 8a/b.
Obsolete types of welded flange jointing.

We will just give two examples[8] (Fig. 8, a and b) to show the unsatisfactory type of butt joint obtained under the old Regulations. In order to effect the transition from a thin to a heavier flange plate, in each case a piece of the heavier plate was cut away in such a manner as to join the thin plate to the thicker one by a V-shaped weld; the remaining part of the thicker plate was then used to reinforce the joint by means of fillet welds. According to present-day views this reduces the fatigue resistance of the V-welds because the root of the welds cannot be re-welded. For reasons given above, the fillet welds even spoil the reinforced joint since the fillets tend to reduce the fatigue resistance of the parent metal. The butt plate or strap shown in Fig. 8b does not entirely compensate for this defect. A case is mentioned in the Report of Board of Administrators in which a butt weld without reinforcing straps on either side proved stronger than the butt weld which had them. It is theresfore advisable whenever possible to use a simple butt weld for the flange plates; if, however, further flange plates have to be added by means of fillet welds, then the flange plates must be flattened or tapered as shown in Fig. 9[9] (see Fig. 27 p. 363). The front fillet weld and the beginning of the side fillet welds must be machined.

[8] *Schaper:* „Feste stählerne Brücken" (Rigid Steel Bridges), Figs. 88 and 89.

[9] The machining mentioned by *Kommerell* in his „Erläuterungen" (Commentary), Part II., P. 73, Fig. 27 V, is not as good and has been superseded by a newer method.

Turning now to the actual execution of the weld joints, endurance fatigue tests carried out with pulsator machines showed that fatigue resistance in all the fillet welds was considerably reduced in those cases in which the weld metal had not penetrated right into the welding root. This proves therefore that it is extremely important to get real inroad when making the weld. In order to obtain this, while simultaneously preventing too much inroad into the parent metal, the welding wire should not be too thick. If the welding wire used at the beginning of the work is too thick, the welder, when trying to obtain good inroad, may run the risk of burning away too much of the parent metal. (See Fig. 10.) When making fillet welds care should be taken not to penetrate into the side walls of the parent metal too far because the change in the texture of the metal

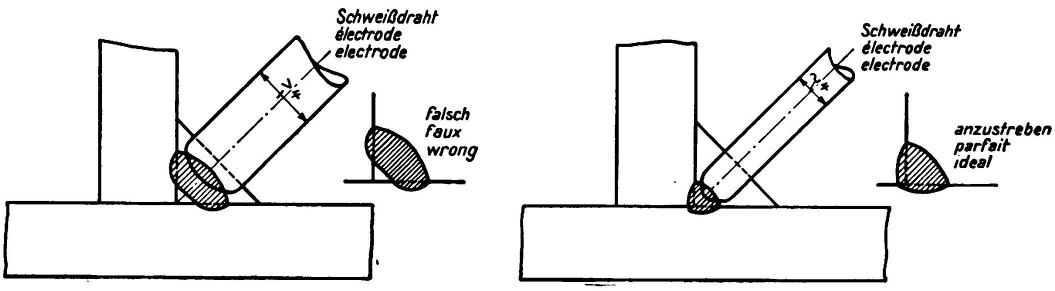


Fig. 10.

weakens the cross section and the result will be reduced fatigue resistance in parts which are dynamically stressed. With structural elements having a maximum thickness of 20 mm, the diameter of the welding rods should not be less than 3 mm.; with sections of over 20 mm. thickness, the minimum diameter should be 4 mm. Current intensity must vary according to the parts to be welded; when welding heavy sections (and this refers also to butt-welding) insufficient current may result in producing welds which do not properly penetrate the parent metal on account of the heat having been too rapidly dissipated. As however, current intensity depends on the cross section of the welding rod, Dr. Dörnen has tried, and succeeded, in using sections  or  for welding rods. Another important point is that the whole of the parent metal must be absolutely homogeneous and there must be no flaws in it, as is occasionally the case with highly silicated steel. When using a defective material of this kind it sometimes happens that some of the welded parts in the zone of penetration get detached from the parent metal, as a result of the contraction that has taken place while welding was proceeding. This is a reason for not welding old structures made of puddle-iron.

I have mentioned elsewhere the importance of obtaining good penetration right into the root when butt-welding. The surge load strength of butt welds with properly welded root of steel test pieces (St 37) was approximately

$$\rho_U = 18 \text{ kg per square mm.}$$

This figure may drop to $\rho_U = 12$ kg per sq. mm if there are defects in the root of the weld. If at all possible, when turning the work round after welding, the root should therefore be thoroughly cleaned and all slag and as much of the weld and the parent metal removed as is necessary in order to obtain an absolutely polished and flawless surface. Not until that has been done should the back side of the root be welded. When starting on butt welding there is no point in using specially thin welding wires, as is advisable for fillet welds. Cases have occurred in which the first layer of the weld fractured because the weld metal was too thin for the heavy section being assembled, and could not withstand the shrinkage stresses. This risk is increasingly greater where the parts to be butt welded offer greater resistance to contraction. Consequently, when working with long structural elements of large section, the first pass of

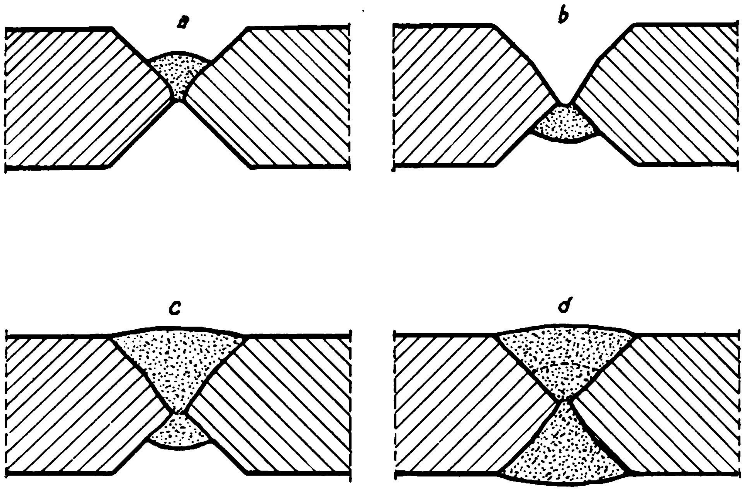


Fig. 11.

a) Welding from one side.
b) Turning and opening-up of root, x-raying.
c) Welding of root and filling same.
d) Turning and filling, x-raying.

weld metal should be tested for cracks by X-ray examination. This is an economically sound proposition because it is cheaper to rectify those defects in the early stages than later on when the joint has been practically filled up. It is of advantage while welding to approach the two elements which are to be assembled; this can be done by means of presses or pulling appliances whilst cooling takes place. The friction between the various parts which have been placed for welding is reduced in this way and the elements can follow the direction of contraction more easily. With thick butt-welded joints it has been found advisable to start by welding o n e side only for a third of its depth, then the work is turned round and the root of the back side welded, after which, as a final operation, the remainder on the first side is filled up. (See Fig. 11.) Heating[10] of the structural elements until the first pass of weld metal is cold reduces risks of fracture. X-rays have proved excellent for investigating butt-welds of thickness of as much as 80 or 100 mm. Thanks to X-ray exami-

[10] See *Bierett:* „Stahlbau" (Steel Struktural Work); 24. 4. 1936.

nation it has been possible gradually to obtain absolutely perfect butt welds; until this method of testing was introduced, demands had been frequently made for the elimination of defective welded seams. Control of quality by means of X-rays has also had remarkable educational effects, as such investigation serves to show welders more clearly what they should aim at producing. When this method became compulsory for testing bridge welds, the large steel manufacturers procured their own X-ray apparatus, then later on other firms, which had at first refused to install such plant, realised its value and the importance of X-ray screening, and are now increasingly introducing the practice of thoroughly examining the welds in the workshop by X-ray investigation so as to avoid the risk of having their work rejected by their customers whose employees would have tested the welds in this way and discovered what defects there were.

C. Stiffeners.

The fact that not only booms of wide, flat steel plates, but also the special sections shown in Fig. 5, a, b and c, had to be more carefully braced and placed in such a way as to avoid any intervening gaps than was necessary for riveted girders became clear from the very first, because a thin weld used for assembling heavy flange plates to the relatively thin web cannot withstand lateral stresses (particularly when these occur in the compression boom) as satisfactorily as a good stiffener obtained by the angle iron of the boom. Furthermore, the distances between web stiffeners had to be shorter in the welded than in the riveted structures. While in early days stiffeners were welded unhesitatingly to both flanges, later on endurance fatigue tests with fillet welds placed vertically to the direction of force and with the zones in which the lateral fillets start and end led to the recognition that, on account of the considerable reduction of fatigue resistance, it would be necessary to prohibit welding of stiffeners and girder connections in the tension flange for bridge construction. As an expedient small pieces of metal were neatly wedged below these parts (Fig. 12, see Fig. 12 V, p. 384) and these may only be welded to the stiffener. At A, from where the stiffeners may be welded to the web, the bending stress in the web may not exceed $\delta = a \cdot \delta$ perm.

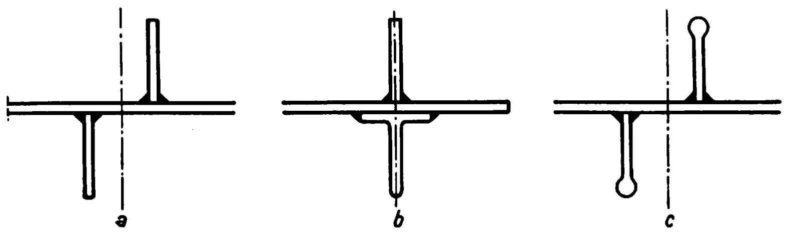


Fig. 13.

The web stiffeners[11] are mostly made of flat steel (Fig. 13a), ⊥ steel (Fig. 13b) or of bulb steel (Fig. 13c), occasionally of I steel or rails. In order to avoid an accumulation of welds on the web, the stiffeners are arranged differently (See Fig. 13, a, b and c), or else flat steel is used on the one side and ⊥ or I steel on the other.

[11] See *Schaper:* „Feste stählerne Brücken" (Rigid Steel Bridges), Page 63.

The seams which connect the stiffeners to the web should not be thicker than necessary. For small bridges a fillet dimension of a = 3 to 4 mm is sufficient. The usual practice is to weld the stiffener to the webs before the latter are welded to the flange; this is done because of the shrinkage stresses which are otherwise set up when welding the stiffeners to the webs.

D. Girders for road decking.

1. Longitudinal girders for decking.

The general rule is that longitudinal beams should have through plates at the cross girder connections. This arrangement is even more important when welding longitudinal beams to the cross girders than when riveting them, because welded joints subjected to similar stressing as rivet heads, the former withstand the stress less well than rivet connections. With regard to the encastré moment, in order to avoid laborious calculations, the German regulations for railway bridges have laid down the following rules:

Table [12]

1	2	3	4
Nr.	Designation	$\gamma' \cdot M_0$	
		St 37	St 52
1	Moment in end bays and at interruptions in the decking	1.0 M_0	1.2 M_0
2	Moment in intermediate bays	0.8 M_0	1.1 M_0
3	Moment at supports of intermediate longitudinal beams	0.75 M_0	0.9 M_0

M_0 ist the maximum bending moment for a simply supported, single span longitudinal decking beam. The coefficients (γ') of the Table, columns 3 and 4, take fatigue resistance into account.

The through plates of the longitudinal decking beams must be dimensioned according to the coefficients given in line 3, columns 3 and 4 of the Table.

The connections must be calculated as for a reaction force of max $A' = 1.2\ A_g + \varphi \cdot A_p)$ Here $\gamma = 1$.

With these through plates the fillets which connect the top boom of the longitudinal decking beams must be considered as interrupted by the cross girders (example: see Fig. 14), so that the fillets which start or end at B and C are welds whose permissible stresses must be reduced accordingly.

If the upper edge of the longitudinal decking beams is at the same level as that of the cross girders, as shown in Fig. 15, then the longitudinal beams and the cross girders may be joined by butt welding and the trough plates may also be welded to the cross beams. In particular, the end fillets on the cross beams which connect the through plates to the cross girders must be so made

[12] For further information see: Kommerell, Commentary, Part II. P. 61. For examples of calculations pp. 107 to 117.

that the passage from weld to parent metal is a very gradual one (light end fillets).

When constructing large bridges in which the decking construction has to be erected on the site itself, it will be found most satisfactory to connect first of all the cross girder situated in or near the middle of the bridge to the main girder, then the longitudinal beams which have to be joined to these cross girders are welded, and the through plates are put in position; thereupon the next cross girders are welded to the longitudinal beams which have been fixed in their final position. The next operation is that of joining the cross girders to the main girders. The work proceeds in this way until the end cross girders have been reached. The last of the longitudinal beams are cut to their requisite length at time of erection. The advantage of this procedure is that shrinkage stresses are kept at a minimum.

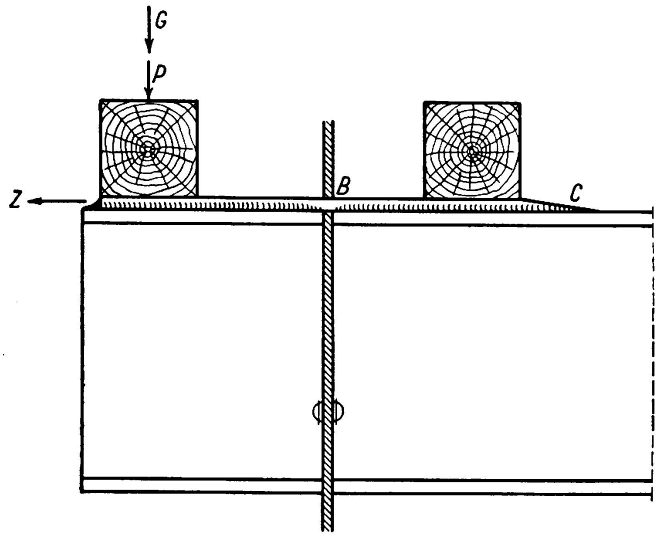


Fig. 14.

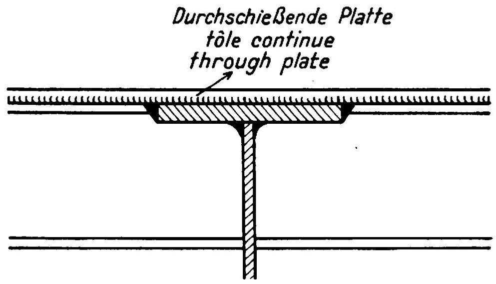


Fig. 15,

2. Cross girders.

The following Regulations govern cross girder connections for railway bridges: The connections between cross girders and main bridge girders are so dimensioned that an encastré moment max M of at least 25 per cent. of the maximum moment of the cross girder can be withstood.

The reaction forces of the cross girders on the main girders must be calculated according to the following formula:

$$\max A' = 1.2 \; (A_g + \varphi \cdot A_p).$$

The encastré moment of the cross girder on the main girder cannot exceed the resistance which the main girder offers to the torsion caused by the deformation of the cross girder. The figure of 25 per cent. was laid down in order to avoid the necessity of making a complicated calculation on each occasion. As a rule this coefficient should answer the purpose[13].

[13] Examples for calculating a cross girder and for computing the torsional moment (Verdrillung), See *Kommerell,* Commentary, Part II., Pp. 117 to 124.

When main girders for bridges[14] are transported in one piece to the site (girders having a length of 61.7 metres and a depth of 3.82 metres have been transported by rail) (Fig 16), welding on the site itself can be avoided by welding the connecting members for cross beams or frames in the workshop,

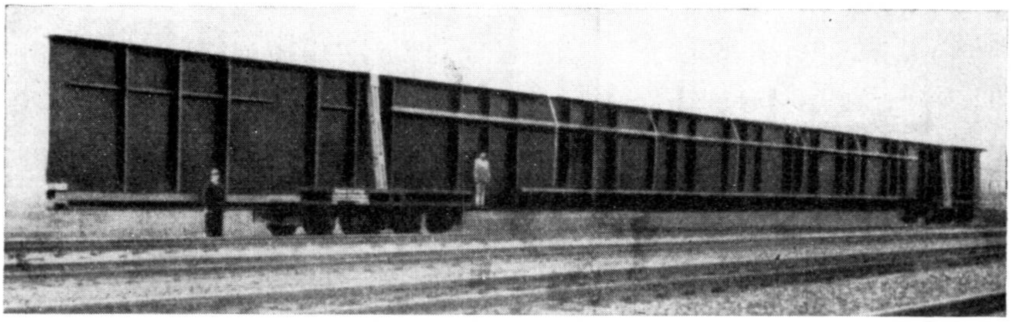


Fig. 16.

while the intermediate parts are fitted into their places at site by riveted joints (Fig. 17). If all the joints have to be riveted at site then it is advisable to place the cross beam joints in the neighbourhood of the zero points of moment (points of counter flection). Here care must be taken to counteract the deleterious effects which may result from shrinkage stresses.

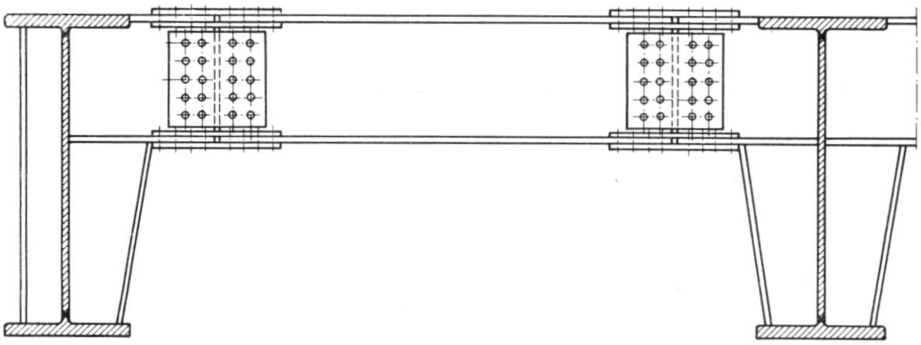
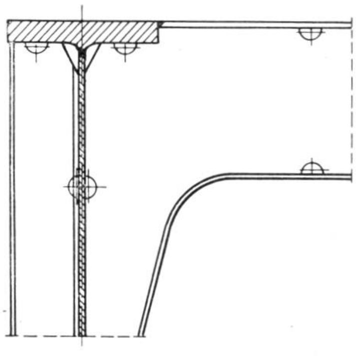


Fig. 17.

Cross beams and frames may be butt-welded to booms which are subjected to compressive stresses only. (See Fig. 18.) When welding booms subjected to alternating efforts — such as may be found over the supports of continuous

Fig. 18.



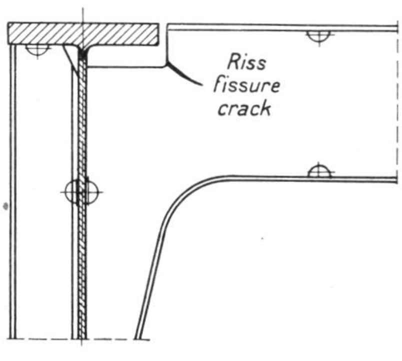


Fig. 19.

[14] See *Schächterle:* „Der geschweißte Vollwandträger", (Plate Girders) in the „Bauingenieur" (Structural Engineering), 17[th] April, 1936, Pp. 135 und 136.

girders — care must be exercised to reduce the permissible stress according to the Regulations.

In spite of notching, cracks have sometimes occurred, as will be seen in Fig. 19. For this reason butt welding is preferable.

E. Wind, Cross, Brake and Anti-Sway bracing.

The following Regulations exist in Germany for these kinds of bracing:

1) These types of bracing must be calculated according to the rules laid down for riveted bridges. The dimensions of the web members, in particular, should be determined without taking into account either the alternating stresses or surge load stresses. The permissible stresses are as follows:

With St 37 $\sigma_{zul} = 1000$ kg/cm^2.
With St 52 $\sigma_{zul} = 1500$ kg/cm^2.

2) The stress in the welded joints of the connections of web members and gusset plates shall not exceed:

$$\sigma = \frac{1}{\alpha} \cdot \frac{S}{F} \leqq \sigma_{zul}$$

that is:

for butt welds . . . . $\alpha = 0.8$
for end fillets or at the point where
the side fillets start . . . . . $\alpha = 0.65$ for St 37
and . . . . $\alpha = 0,55$ for St 52

Alternating stresses and surge load efforts receive the same consideration as in the case of riveted bridges, namely by reducing the permissible stress in the bracings to $\sigma_{zul} = 1000$ kg/cm^2 for St 37 and $\sigma_{zul} = 1500$ kg/cm^2 for St 52.

3) If the gusset plates are welded to the booms then it will be necessary to reduce the tension in the latter to correspond to the α-coefficients.

In certain cases it may be preferable to rivet the gusset plates of the bracings to the booms (weakening of the tensile boom by rivet holes). If the web members of bracing systems were welded to the gusset plates, it would be impossible to get a clear and comprehensive picture of the stresses because of the shrinkage stresses, and it is therefore advisable to rivet the web members of the bracings to the gusset plates.

Certain structural parts — for instance, stiffeners or girder connections — may be fillet-welded to the tensile area of the web plate only in that part where the maximum bending moment stresses in the web are more than:

$$\sigma = \alpha \cdot \sigma_{zul} \quad \text{(Fig. 20)}$$

The distance x from the neutral axis will be:

$$x = \frac{h}{2} \cdot \frac{\alpha \cdot \sigma_{zul}}{\sigma_{zul}} = \alpha \cdot \frac{h}{2}$$

The coefficient α is seen in Tables 2 V and 3 V, lines 14 and 16 of the Regulations.

This prescription represents an innovation and a very important one. It was a well known fact[15] that stiffeners should not be joined by welding to the tension flange. Endurance fatigue tests carried out in the Government Test House at Dahlem with a welded plate girder in which the stiffener extended only partly into the tension zone (Fig. 21) showed that here care must be exercised as the fatigue fracture started at point A (zone in which the fillet weld starts).

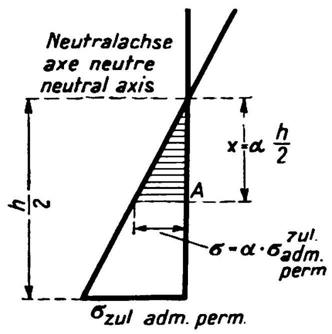


Fig. 20.

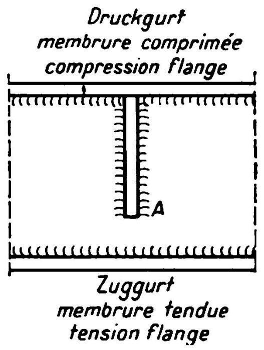


Fig. 21.

The material test Houses represented on the Working Committee did not attach much importance to the stiffeners or girder connections in the compression flange being welded to the flange. This is significant as otherwise the design and arrangement at the connecting point between flange plates and web would be very complicated if dealing with deep girders and heavy lateral forces. In many cases small wedge-shaped plates should have been applied between flange and stiffener.

F. Model Plan for a welded plate girder railway bridge.

The new Regulations issued by the German State Railways concerning the welding of plate girder railway bridges have been in force for over half a year and a large number of bridges have been constructed in accordance with them, and now plans to serve as samples when designing welded plate girder railway bridges are being drawn up, just as was done earlier for riveted bridges. Fg. 22 shows the main parts of a plan (incidentally not yet approved) of a plate girder bridge of unlimited constructional height and continuous decking of 18 metres span. The height of the girder is about one tenth of the span, the main girder interval is 2.50 metres and that of the crossbeams with seven bays about 2.57 metres. The flanges of the main girder are of one thickness and without joint. Reference has already been made to the possibility of easily X-raying the butt weld between flanges and web when flat ridged steel is used. The upper flanges of the framed cross beam are connected to the flanges of the main girders by butt welds. Small plates are also placed between the lower flanges of the main girder (tension flanges) and the cross beams, and

[15] *Kommerell:* „Erläuterungen . . ." (Commentary on the Regulation for welded steel structures). Fourth Edition, Part I., Structural Engineering, P. 51, Berlin, 1934, Published by Wilhelm Ernst & Son.

also between the lower flanges of the main girder and the stiffeners; these plates must be very carefully fitted into place after welding. The stiffeners of the main girders at the cross beam connections are made of I P—16 and those

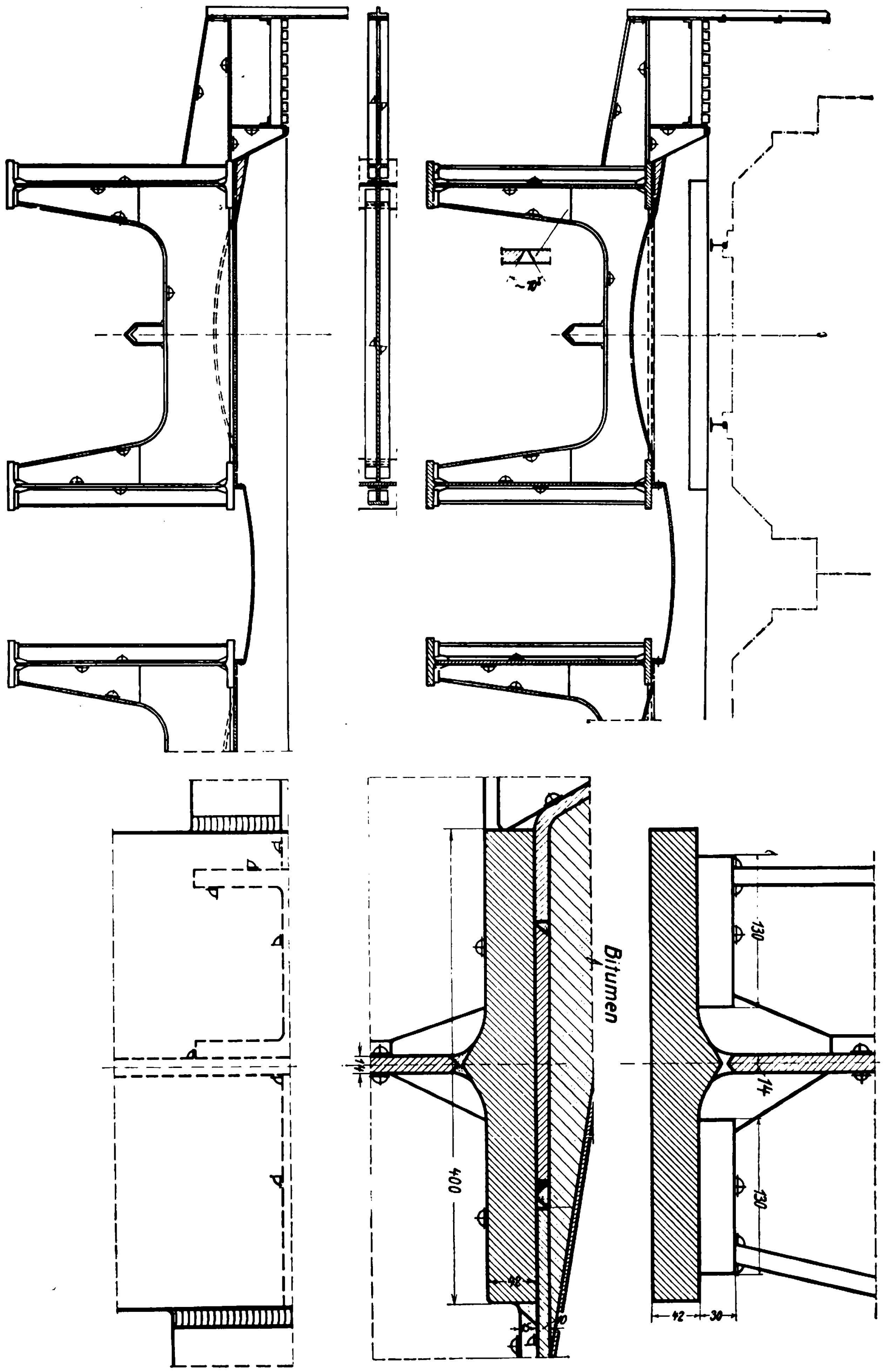


Fig. 22.

in the centres of the bays of I—34 cut in halfs and flat steel. The dimensions given in the plan comply with the new Regulations and the plan itself is based on previous experience.

III. Strengthening of old riveted lattice bridges by welding.

A. Reinforcement of joint in a bottom boom member.

The welding carried out in 1932 was intended to strengthen the apparently inadequate covering of the web joint by welding-on supporting straps ▭ 50.15 to it. This reinforcement no longer corresponds to the knowledge gained by endurance fatigue tests since carried out. There is reason to believe that after some time fatigue cracks will occur at the extremities of these reinforcement straps, most probably in the lower flange member which is not reinforced. The cause of this is that the points where the lateral fillets start and end and the head fillets are situated obliquely to the direction of force. In any case it cannot be expected, that 2×10^6 loading repetition will be withstood with altern-

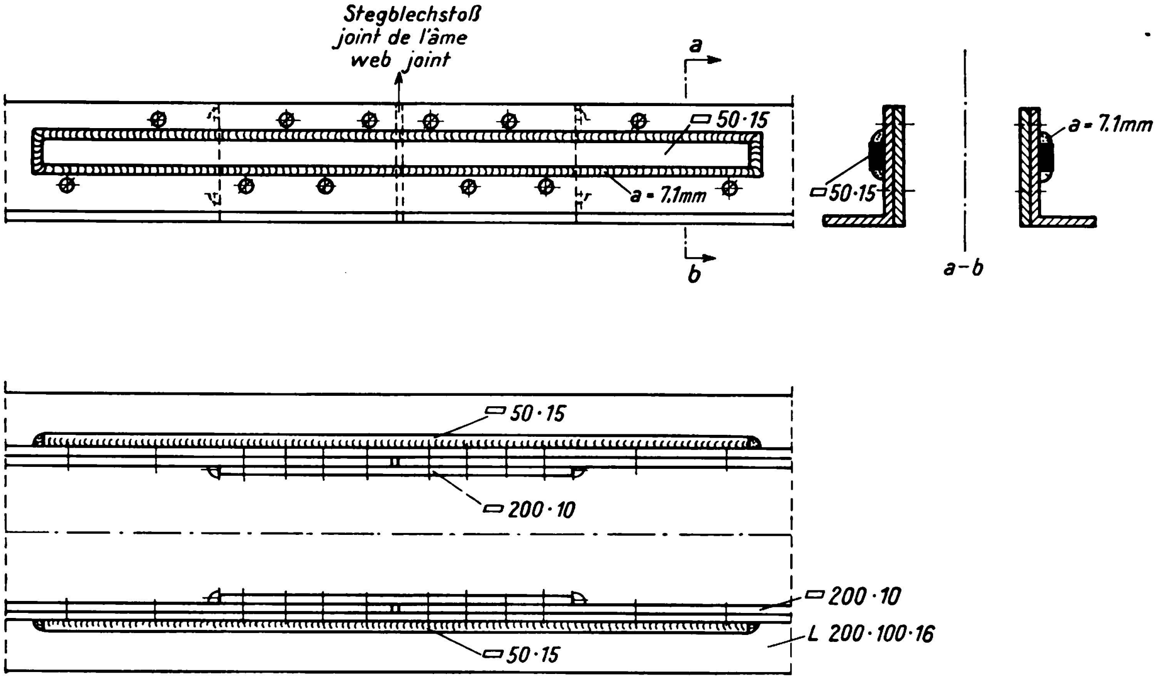


Fig. 23.

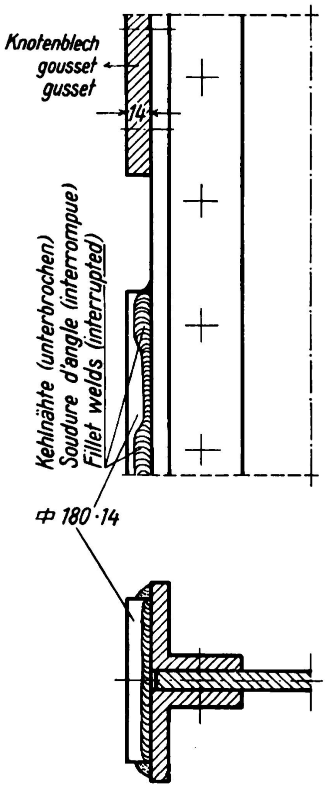
ating stressing up to the permissible stress as would be the case with a new bridge. Present-day experience goes to show that the reinforcement straps which have been welded to the web would have to be carried into the panel points. The reinforcement could be slightly improved by machining the ends of the side fillets and the end fillets in order to produce a smooth and gradual transition from weld to plate. But even so, and in the most favourable circumstances, the life of the bridge would be increased by a few years only. (Fig. 23).

B. Reinforcing of verticals of a lattice bridge.

In 1931 all the posts of the bridge were strengthened by welding plates ▭ 180.14 to them in order to increase resistance to lateral deflection of the compression booms of the bridge which was open at the top. As a rule strengthening plates of this kind end about 10 cm below the lower edge of the top boom gusset plates which latter have a thickness of 14 mm (Fig. 24). At that time the reinforcement plates were welded on both sides of the struts by means of a series of lateral fillets, and narrow tight joints were welded between the fillets and at the one end. No defects have been recorded for these plates.

In the case of two of the posts the reinforcement plates ▭ 180.14, generally of the same length, were nearer to the lower edges of the gusset plates which were larger at this point (Fig. 25, A). As there was no possibility of making a

Fig. 24.



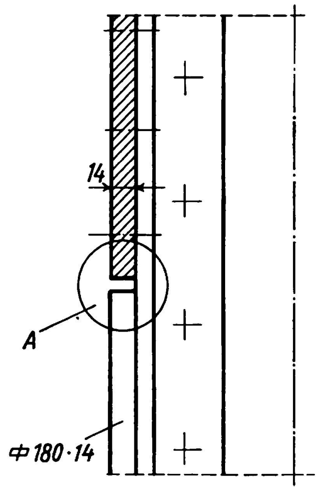


Fig. 25.

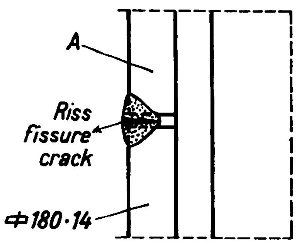


Fig. 26

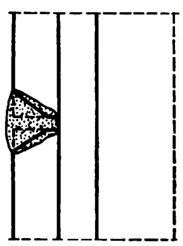


Fig. 27.

thin fillet weld, the gap A was merely welded over (Fig. 26). This weld cracked at one point.

As this reinforcement plate ▭ 180.14 was joined to the gusset plate, the stresses set up passed direct from the reinforcement plate to the gusset plate. Probably the notching action of the gap beneath the weld caused the crack, due to dynamic stressing[16].

If it was absolutely necessary to weld the reinforcement plate to the gusset plate, then the gusset plate edge should have been bevelled in order to obtain a satisfactory V-weld, and further, the extremity of the reinforcement plate ought to have been prepared with a view to making a V-weld (Fig. 27).

The defective joint should have been removed and replaced by a V-weld complying with requirements.

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[16] Re. Bad effects of notches, see Kommerell, Commentary to Regulations concerning the welding of steel structures, Part. I, 1934, P. 39, Item c, and illustrations 13 and 14.

Reinforcing of riveted bridges by welding.

C. Reinforcing of struts of a lattice bridge.

The following example shows how important it is when strengthening struts to ensure a perfect flow of forces from the strengthening elements to the panel points:

The struts of the lattice bridge shown in Fig. 28 were reinforced by welding in the winter of 1930 to 1931. At that time no experience was available on which to base the work, and the first Welding Regulations did not appear until

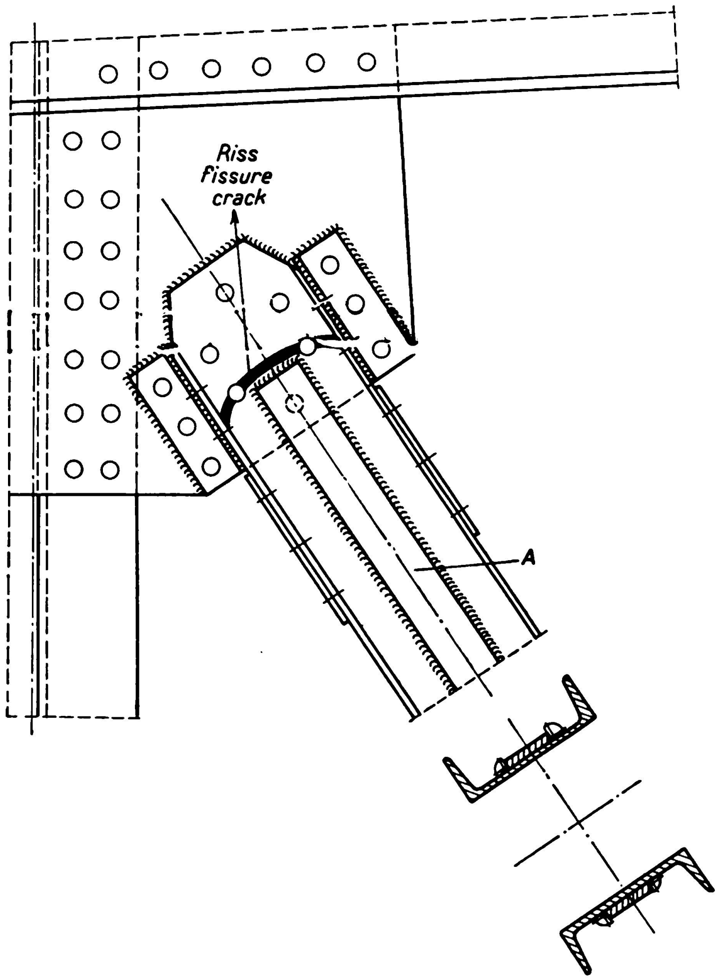

Fig. 28.

May 1931. Flat bar iron reinforcing straps A, welded on both sides of the webs of channel section iron terminated behind the previously countersunk rivet of the connection (Fig. 28). It was thought that the connection had been sufficiently reinforced according to the calculations which had been made by connecting the end of the member and the angle irons to the gusset plate by fillet welds and by welding the projecting leg of the angle iron to the projecting flanges of the channel section iron.

Three quarters of a year after the strengthening had been effected, the web of the channel iron broke just in front of the end fillet of the reinforcing plate. This is shown in Fig. 2.

The causes of this fatigue fracture would appear to be the following:

a) An accumulation of stresses in the broken cross section of the channel iron because, according to this solution, the forces of the strengthening strap passed in the first place through the web of the channel iron which was already subjected to tension instead of passing immediately to the gusset plate.

b) Notching action of the end fillet weld, increased by the rivet hole weakening of the channel iron situated in the same cross section.

c) High shrinkage stresses. As the member was firmly riveted while welding proceeded, high shrinkage stresses occurred while the welds were cooling, these, however, could have been attenuated by heating the member during the process of welding. Care, however, must be exercised in this con-

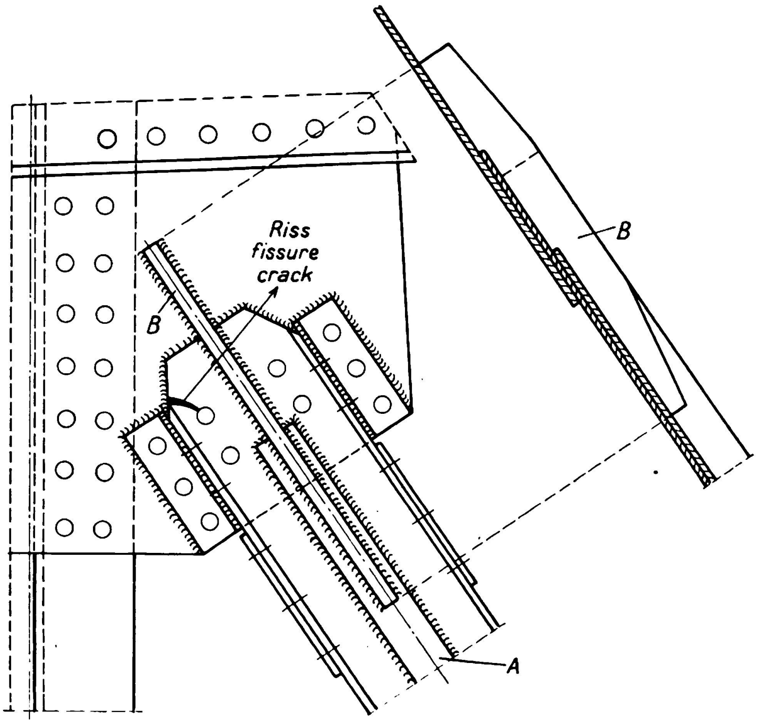

Fig. 29.

nection because the member is subjected to stress resulting from the weight of the bridge itself. The shrinkage stresses due to welding cannot be easily determined because Hook's Law no longer holds good for plastic material on account of the change of the modulus of elasticity, and so the actual conditions governing tension are very far from clear.

The connections of the other diagonal struts of the bridge were corrected later by removing the defect described above. (This is shown in Fig. 29.) The original reinforcements were connected to the gusset plates by means of flat iron straps on edge B.

In the spring of 1936, that is, after the bridge had a further service period of four years and five months, cracks appeared more or less simultaneously on five diagonal struts (see Fig. 29) which connected one corner of the channel iron formed by the tooled flange and the remaining web of channel iron to the nearest rivet hole.

Notching action caused these cracks, because at this point the forces of the projecting channel iron flange pass into the web. It is obvious that notching action is far more prejudicial to welded than to riveted structures, because the angle iron and channel section iron and the gusset plate are connected in an absolutely rigid fashion only after the reinforcing welding has been carried out. Hat it been possible to carry out the reinforcement as far as the gusset plate in the first instance and also to eliminate the welding of the neighbouring angles and the ends of the members, probably the damage might possibly have been avoided.

According to the present standard of welding technique, it would be preferable to connect the two][iron sections of the struts by a web which, without greatly affecting the existing rivets of the connections could easily be extended as far as the panel point.

It is significant that the cracks due to notching action shown in Fig. 28 occurred after the bridge had been in commission for three quarters of a year, while those shown in Fig. 29 did not occur until after four years and five months' service.

Thus it will be seen that when riveted bridges are strengthened by welding it is necessary to exercise great care. The whole position would be clearer if the bridge had been shored up before welding and the member disconnected at least at one of the panel points until after cooling. That, however, would signify that the main advantage aimed at by welding would be lost.

D. Stiffening of slack diagonal struts by welding.

The diagonal members consisting of two flat irons of an old riveted bridge were strengthened by welding and rigidity thus obtained. Figs. 30 and 31

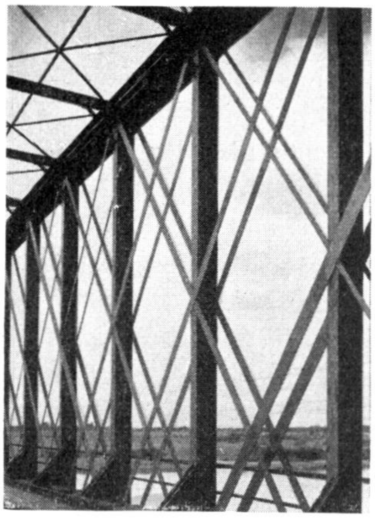

Fig. 30.

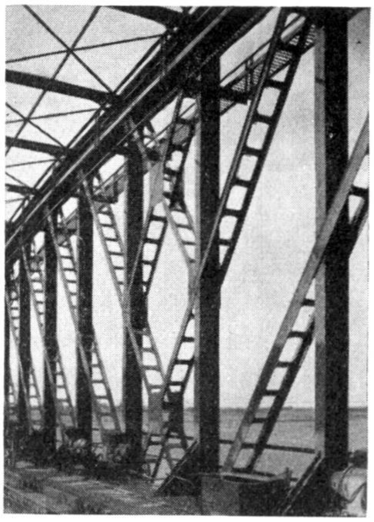

Fig. 31.

represent a bridge in which webs were welded between the iron of the tensile struts into which holes had been cut before welding in order to reduce weight.

In this connection care should be taken to carry the stiffening webs as far as possible into the panel points, and the lateral fillets which terminate at that place should be very carefully machined so as to reduce risks of notching action (Gradual transition). By welding these strengthening webs into the structure, the diagonal struts which had been slack fortunately became more taut. Nevertheless reinforcement is merely an expedient aiming at extending the life of a bridge by at least a few years. The statements concerning stress conditions made in connection with the earlier example apply here equally well.

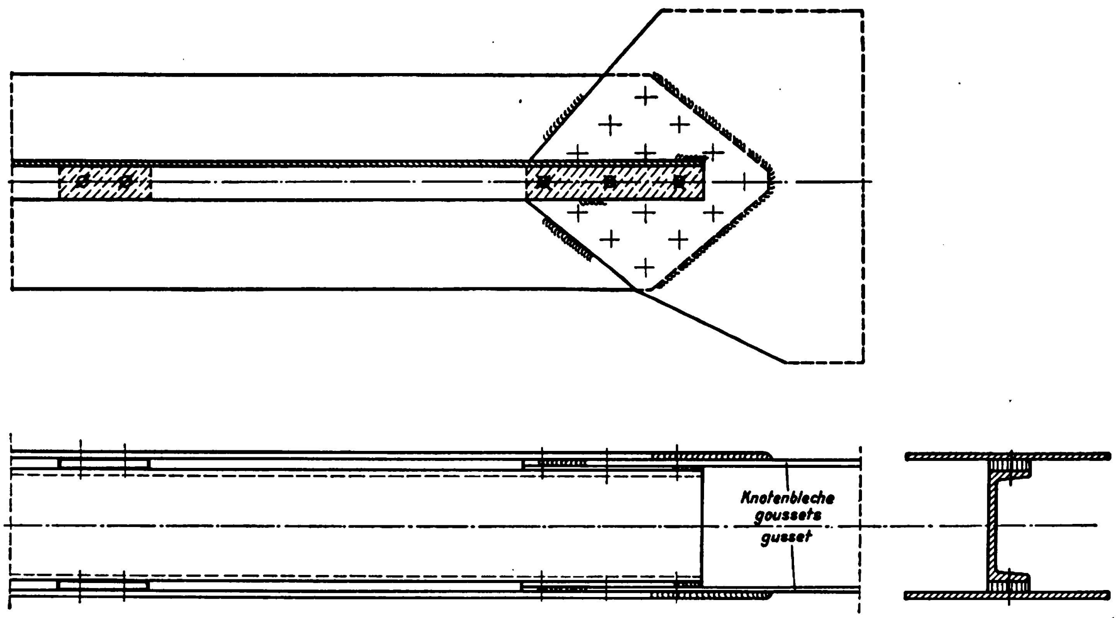

Fig. 32.

In Fig. 32 another old bridge is seen. In this the flat iron struts were reinforced by riveting channel section iron to them. Welding was applied solely to strengthen the rivet connection at the panel point, as the gusset plate had already been very considerably weakened by the drilling of rivet holes, and the channel section iron could not be extended further into the panel point. This work was carried out in 1931, and as at that time no practical experiments had been made, the solution was considered acceptable. Present-day opinion. however, is opposed to the further stressing of the existing tensile struts and their connections by welding the strengthening element to them. (For defects see Paragraph C.) The reinforcing element, in this case channel iron, must be connected direct to the gusset plate and the packing plates to the joint, before the bridge is put into commission. Even if no defects have become apparent so far, it may be assumed that the accumulation of stresses and the notching effects of the fillet welds will cause the life of the bridge to be limited.