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Influence of the Form of Welded Connections to Strength and Resistance.

Einfluß der Gestalt der Schweißverbindung auf ihre Widerstandsfähigkeit.

Influence de la forme des assemblages soudés sur leur résistance.

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The opinions as to the best design of welded joints, especially the joints which are mainly subjected to repeatedly recurring stresses, have been considerably modified in all countries since 1931. This change has also considerably affected the design of welded structures, their structural detailing and manufacture. Down to the period mentioned, knowledge and experience regarding the fatigue strength of structural elements were disregarded in the regulations[1]. When, in 1931, some tests had shown[2] that the usage obtaining at that time for the execution of welded joints for machines and bridges could only be partly, if at all, adhered to, one of the chief problems of the welding engineer was that of affording the designing and supervising engineer the data to enable him to design in a form suitable for welding, and to create structures which are suitably dimensioned and can be built expertly.

The problems which confronted us, and still do confront us in this connection, are roughly as follows:

1) How should the weld as such be constituted if, in the form of butt-welds, front-fillet welds, and side-fillet welds, it is to stand up to frequently recurring stresses or to static stresses up to the limit of what is possible at present?

2) What type of weld (butt-weld, side-fillet weld, etc.) is specially adapted for taking frequently recurring loads (tension, compression, alternate tension and compression, bending, shear, etc.)?

3) How are the results of 1) and 2) to be applied when designing structural elements, such as connections of tensile members, joining girders, strengthening of girders by boom-plates, or when joining cross girders to main girders, etc.?

4) What is the significance of the stresses which arise during and after welding?

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[1] cf. *Graf:* Die Dauerfestigkeit der Werkstoffe und der Konstruktionselemente (The fatigue strength of material and structural elements), Verlag Julius Springer, Berlin, 1929.

[2] cf. inter alia, *Graf:* Dauerfestigkeit von Stahlen mit Walzhaut, ohne und mit Bohrung, von Niet- und Schweißverbindungen (fatigue strength of steels with rolling-skin, with and without perforation, of riveted and welded connections), VDI-Verlag, Berlin, 1931.

In the light of our present knowledge, it is certainly possible to answer the questions fundamentally, but there are many partial problems often confronting the designing and supervising engineer which have not been dealt with sufficiently to enable the knowledge to be applied as it stands.

a) Re question No 1.—

Several older fatigue tests showed that structural elements which have to stand up to frequently recurring loads should be designed with gradual transitions of cross-section, so as to obviate the occurrence of tress-peaks where possible. Butt-welds stressed in tension or compression must therefore be expected to be inferior when, as in Fig. 1, they have edge notches or notches in the root of the weld or flaws in binding of the weld metal. The test, of course, shows that this conception is correct, so that less porous welds with re-welded roots and gradual transition give much higher fatigue strengths than welds as in Fig. 1.

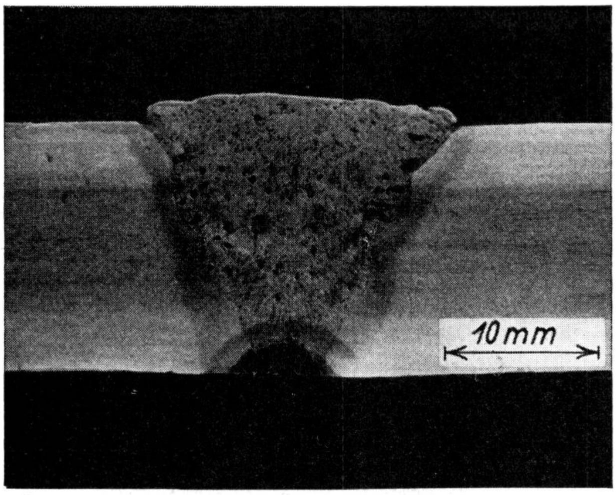

Fig. 1.

The influence of the form of the weld was also found with fillet welds. Welds as in Fig. 2 are inferior, welds as in Fig. 3 superior in quality[3].

As a result of these tests, it was felt that the execution of high-strength welded joints should be made to depend on (a) a suitability test of the welders employed, and (b) the materials it was proposed to use. The men employed on the job must know how to make welds expertly. *As regards the materials used; viz, weld metal (electrodes) and structural materials (and especially high-strength materials), an independent authority must prove that good welded joints, inside and out, can be made from them. The suppliers must also state definitely how the material must be built, and how the electrodes ought to be handled so as to guarantee the turning out of good welds.*

Those responsible for guaranteeing that highly-stressed welds are strong enough to stand the loads to which they are subjected must have the necessary aids for demonstrating the quality of the welds; since only by constant tests on finished welds can they judge as to who is capable of reliably making

[3] cf. inter alia, *Graf:* „Der Stahlbau", 1933, p. 81 et seq., also Zeitschrift des Vereines deutscher Ingenieure, 1934, pp. 1423 et seq.

satisfactory welds. They must make sure that the men doing the job take full responsibility for its satisfactory performance.

The methods available for testing the quality of welded joints have been considerably improved; the equipment for X-raying welded joints has become more efficient and cheaper, so that it is now possible to stipulate that highly stressed, important butt-welds should be X-rayed before the parts are delivered[4].

b) As regards the second question — *which type of joint should be preferred under the manifold practical conditions?* — more definite information is available than is the case with the first question, since it seemed more imperative to develop proper fundamentals for the design of welded structures.

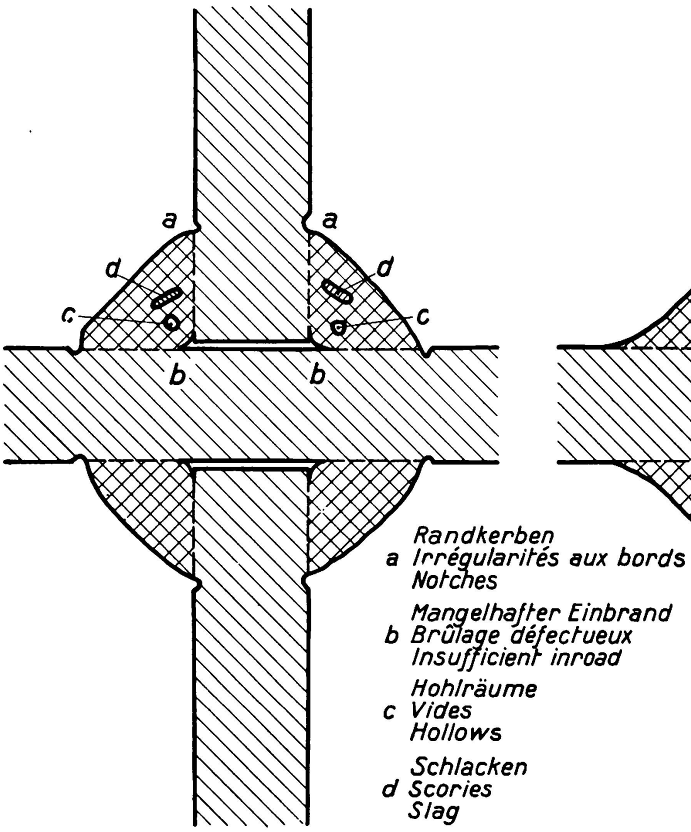

Fig. 2.

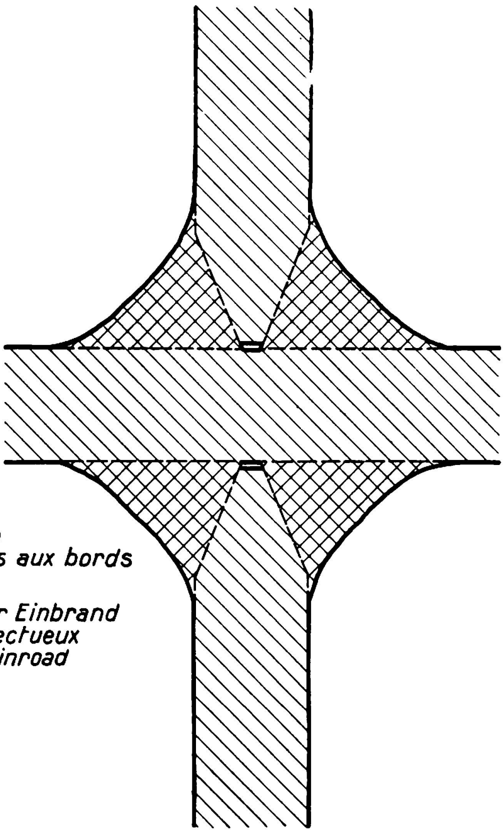
Fig. 3.

Attention was and is being drawn to the fact that the strength of side-fillet welded joints under tensile stresses is not affected by welding flaws to nearly the same extent as the carrying capacity of butt-welded joints. The preparation work necessary is also said to be less with side-fillet welds than it is with butt-welds. In view of these findings, the preference was certainly generally given previously to the side-fillet welds. It should be specially noted, however, that expertly made butt-welds of good quality stand up much better to frequently recurring loads than side-fillet welds do, because high stresses always occur at the beginning of the side-fillet welds — stresses, at all events,

[4] cf. Vorläufige Vorschriften für geschweißte, vollwandige Eisenbahnbrücken (Preliminary Specifications for welded Plate Girder Bridges), Deutsche Reichsbahn-Gesellschaft, Berlin, 1934, pp. 11 and 13.

which are higher than those occurring in butt-welded joints[5]. As a result, breakage takes place on the fatigue test under surge load stress as in Fig. 4. A comparison with Fig. 5 which refers to the simple tensile test on the same specimen and can be used for ascertaining the strength under static load, shows that the tensile test does not offer any information as to the extent of the stress peaks. Figs. 4 and 5 demonstrate that a *side-fillet weld may preferably be used where it has to stand up to static loads, but not to live loads.*

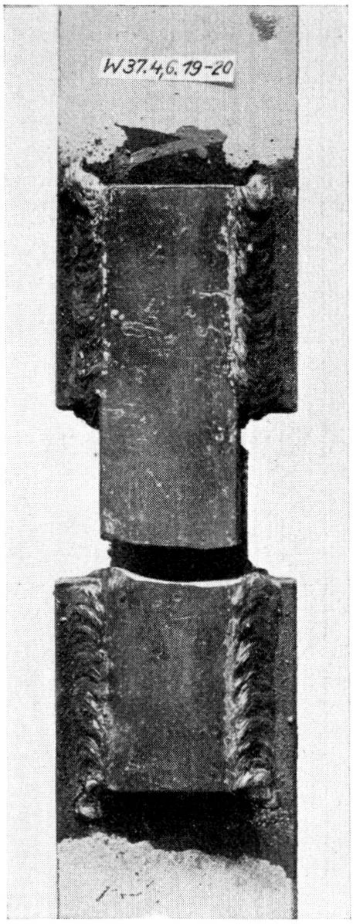

Fig. 4.

Side fillet weld connections after fatigue test.

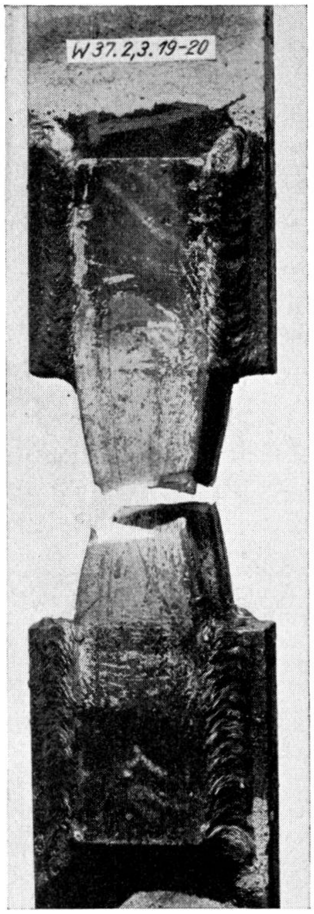

Fig. 5.

Side fillet weld connection after rupture test (dimensions before testing the same as for Fig. 4).

The development of welding transformers, the use of suitable electrodes, the more thorough training of artisans and engineers, progress in methods of testing, etc. have improved the execution of good quality butt-welded joints to such an extent that it is now possible to guarantee the quality of butt-welds made in well managed shops.

With this general discussion as to the suitability of the type of joint and the practical scope of the several types of welding, an investigation was necessary as to whether the strength of the joints when subjected to frequently recurring tensile loads was really satisfactorily determined by ascertaining the surge load strength. Tests were therefore made to ascertain whether, on frequent alterna-

[5] cf. inter alia „Stahlbau", 1933, pp. 81 et seq.

tions between tensile and compressive stresses of the same magnitude, or under the simultaneous effect of static and frequently recurring tensile loads, the amplitude of stress which can be endured is the same or approximately the same as the surge load strength. Many tests[6] showed that the surge load strength is identical with the endurable amplitude of stress for the practical stress range. It was then suggested that the following simple guiding principles[7] be adopted for the dimensioning of welds:

a) For static loads and the total loading the yield point of the material is decisive.

b) The live-load (dynamic) is governed by the amplitude of stress which can be stood up to for frequently recurring loads, and which, for the sake of simplicity, can be determined from the surge load[8].

Fig. 6 is an example of the results obtained from tests on a butt-weld. The strength to withstand the stationary and the total loading is defined by the tensile strength and the yield point. The stress amplitude which the material can stand for frequently repeated loads, has been ascertained as $S = 14{,}5$ kg/mm^2 based on surge load strength and $S = 13{,}1$ kg/mm^2 for the yield point.

This enables the permissible stress to be selected for the static load and the total load, and independently of them, the permissible stress for the frequently recurring load. If, for instance, the permissible stress has been fixed at 0,8 times the ascertained strength, then, in the case of Fig. 6, the permissible stresses would work out at:

a) For static loads and for the total loads: $37.8 \cdot 0.8 =$ approx. 30 kg/mm^2 and

b) For frequently recurring loads: $14.5 \cdot 0.8 =$ approx. 11 kg/mm^2.

The maximum permissible stress would therefore be 30 kg/mm^2. Of this maximum stress, 11 kg/mm^2 may be set up by live loads.

Simple tables could then be drawn up for the designing engineer, showing first. the permissible tensile stresses for live loads, and then the maximum stresses for static and dynamic tensile loads. The maximum stress under static loads is governed by the grade of steel; and the particular type of connection can be ignored provided certain minimum conditions for the execution are satisfied. For defining the permissible stresses set up by frequently recurring loads, a system of grading is particularly necessary in terms of the type of joint (butt-welds, side-fillet welds, front-fillet welds).

The conditions are simpler still with joints stressed in *compression*. In this case it is obvious that the butt-weld should usually be preferred for taking up compressive stresses. As a rule, it is not difficult to make butt-welds which enable static compressive stresses to be transmitted in the same manner as in unjointed members. Where frequently recurring compressive stresses have to be taken up, it should be noted that peak stresses occur at sudden transitions of cross-section, i. e., the edge notches of butt-welds, at notches in the root of such welds, etc.[9] and these may lead to local permanent deformations at the

[6] cf. inter alia, „Stahlbau", 1933, pp. 92 et seq.; „Stahlbau" 1935 pp. 164 et seq.

[7] cf. „Stahl und Eisen", 1933, pp. 1218 et seq.

[8] Here and in other parts of the discussion, this is understood to mean the original stress which the weld will stand after 2 million reversals.

[9] cf. „Stahl und Eisen", 1933, p. 1219.

upper limit of loading and when relieved from loads, tensile stresses are set up at the base of the notch. This means that, under frequently recurring compressive loads, transverse cracks are set up in untreated butt-welds; the same type of cracks are also set up under frequently recurring tensile loads, but at much higher total stresses[10] for compressive loads. It may be assumed that the surge load compressive strength of butt-welds of proper workmanship lies roughly at the compressive yield point (crushing point) in the case of Steel 37,

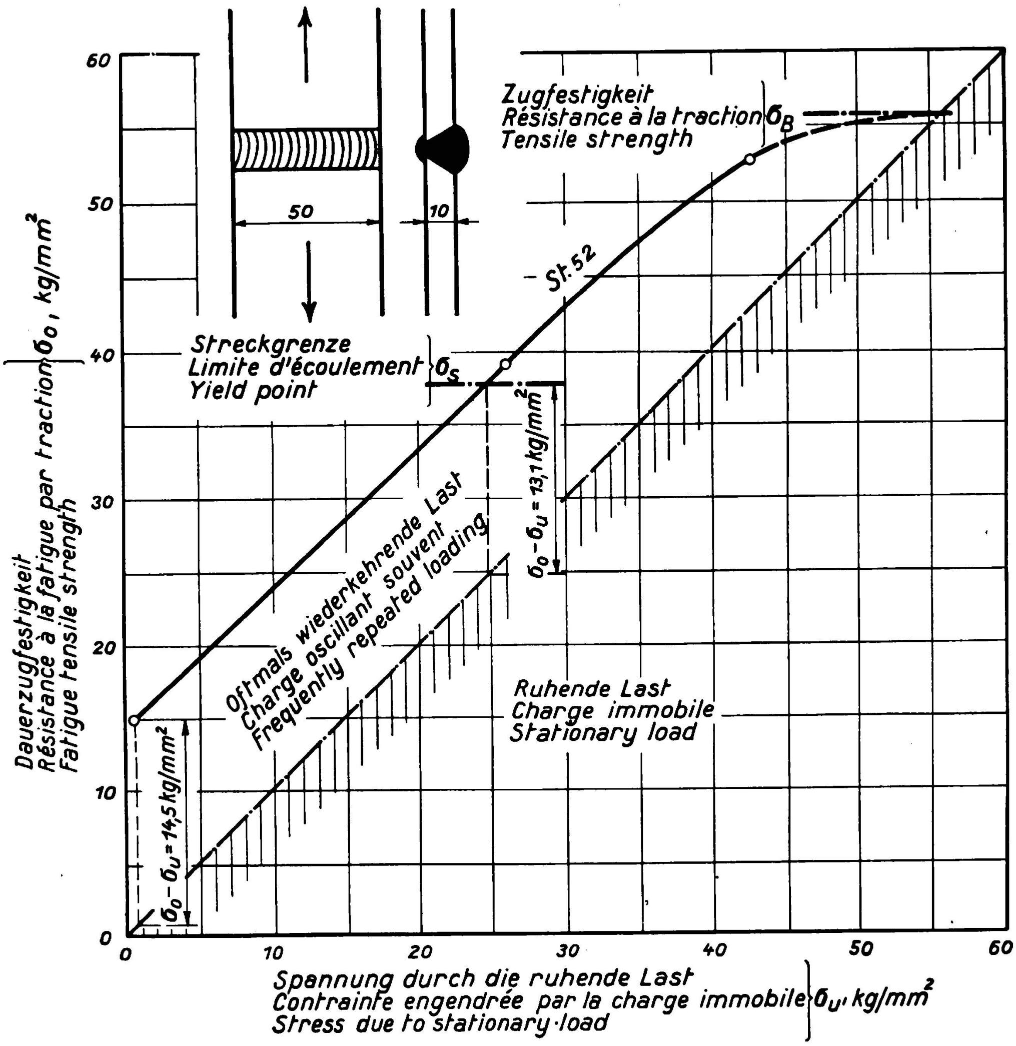

Fig. 6.

for which reason the permissible compressive stress of parts made from Steel 37 can be made the same with and without butt-welds within the usual limits obtaining.

Further observations finally led to the assumption that the results of the tensile and compression tests also apply to the tension and compression zone of rolled steel girders[11].

[10] cf. „Stahlbau", 1936, pp. 71 and 72.

[11] cf. inter alia „Stahlbau", 1934, pp. 169 et seq. Tests with rolled girders joined with butt-welds, etc. will be reported on separately.

An investigation also had to be made as to the dimensioning of side-fillet welds and front-fillet welds when the stresses coming on to them are shearing stresses. In side-fillet welded joints, as in Fig. 4, of varying length of weld and subjected to frequently recurring tensile stresses, the surge load shearing strength could be estimated at 10 kg/mm^2 at least[12].

When the members joined by side-fillet welds were subjected to frequently recurring compressive stresses, the surge load shearing strength was found to be roughly 12 kg/mm^2.

If the welds of the side-fillet welded joints were dimensioned in accordance with these results, the welds would be found to be smaller than what is required at present. This result cannot be applied as it stands to joints subjected to frequently recurring loads, since the dimensions of the welds affect the stress-peaks at the beginning of the weld. For this reason, it would seem necessary to limit the ratio of the shearing stress of the weld to the stressing of the jointed members. For parts as in Fig. 4, the surge load tensile strength has increased with decrease in $\rho : \sigma$. For $\rho : \sigma = 0.5$, the maximum surge load tensile strength was almost reached[13].

c) The third question, viz., the application of the fundamental results to isolated problems requiring solution, can only be answered step by step, since there are technical and economic limits to the design of the joints.

As regards the use of side-fillet welded joints, two compromises are possible: (1) These joints are not so suitable for standing up to frequently recurring loads, and a correspondingly lower permissible stress should be adopted for them than for butt-welds; or (2) by selecting suitable material, or by altering the design and construction of the joint with the object of reducing the peack-stresses. Endeavours have been made to increase the carrying capacity of side-fillet welds by adopting a special form of weld and by distributing the weld metal in different ways; e. g., by making a gradually increasing thickness of weld, by varying the thickness and length of the side-fillet weld, by adopting welds of different cross-sections. Experience shows that not much can be achieved by these means in the case of steel structures. More can be done by specially selecting the cross-sectional shapes of the joined members and by arranging the welds to suit. With members of rectangular section stressed in tension, the surge load tensile strength was raised by using flat sections 50×16 mm, or square sections, in place of 80×10 mm flat sections, und using short heavy side-fillet welds[14]. Much higher surge load tensile strengths were obtained when the joints were made with [-sections and front-fillet welds, while the highest surge load tensile strength, viz., 12 kg/mm^2, was obtained with joints as in Fig. 7[15].

It will therefore be noted that the difference in carrying capacity of the most important welds used for frequently stressed tension connection in steel structures — i. e., (a) the raw butt-welds, and (b) the raw side-fillet weld connec-

[12] „Bautechnik" 1932, p. 415.

[13] „Bautechnik", 1932, p. 415; also Fatigue Tests with Welded Joints, (Dauerversuche mit Schweißverbindungen), VDI-Verlag, 1935, p. 25.

[14] Zeitschrift des Vereines deutscher Ingenieure, 1934, p. 1424.

[15] Zeitschrift des Vereines deutscher Ingenieure, 1934, p. 1424.

tions — has gradually become smaller. Whereas the surge load tensile strength can be put at roughly 18 kg/mm^2 for good butt-welds, figures of up 12 kg/mm^2 may now be expected for properly designed and well made side-fillet welds as in Fig. 7. The carrying capacity given for the butt-weld can only be guaranteed where an X-ray examination has proved it to be satisfactory. X-raying is not necessary in the case of the side-fillet welded joint, provided it has been made by careful and skilled welders.

In view of the above remarks, it will readily be unterstood why, previously, when butt-welding could not be reliably carried out, it was attempted to counteract any possible defects in butt-welds by *cover plates* placed over the joints. By doing this, the carrying capacitiy under static load is ensured, so that the strength of the joined section could be made full use of in tensile tests. Where, however, frequently recurring loads had to be transmitted, the ordinary cover plate only improved the joint when the butt-welds were bad. With good butt-welds subjected to moderate stressing fracture occurred as in Fig. 8 because high stress-peacks formed at the front edge of the cover plate, due to a sudden change in cross-section. It was therefore necessary to find a type of cover plate which gives the same carrying capacity as a good butt-weld alone does. This result was first obtained with cover plates treated as in Fig. 9. In the case of Fig. 9, the cover plate (a) is chosen as wide as possible, fixed at the front edges with thick fillet welds (b) with a stady and gradual transition, secured laterally with thinner side fillet welds, then machined locally so that all notches at (c) which may matter in the region of stress transmission are eliminated[16]. Of course good quality electrodes must be used if these conditions are to be complied with.

The observations as to the shape, etc. of the cover plates apply, by analogy, to the ends of flange plate strenghtenning of girders. A special report will be presented dealing with tests in this connection. When web stiffeners are fitted, it should be noted that the tension boom is less able to stand up to frequently recurring loads when the stiffeners are welded to the tension boom[17]. The welded-on stiffeners may also diminish the strength in the web.

Investigations have also been made into the strength of fillet-welds which have to stand up to a bending moment in addition to a shear force.

As regards structural work, tests have been carried out at Dresden[18]. For bridges, information is derived by testing large models such as in Fig. 10. In this connection, the stressing of the cross girders and the main girders should be arranged as provided for in the regulations in force at present, or as suggested for future regulations. In the case of Fig. 10, the connection has fillet-welds on all sides and the usual stiffening arrangement. Rupture took place in the cross girder for stresses approximating to those obtaining for girders of the same simply supported type. Another remarkable fact is that a crack as in Fig. 11 occurred in the web of the main girder before the cross girder fractured,

[16] The detailed report in this connection to the German Commission for Steel Structures is in preparation.

[17] cf. *Schulz* and *Buchholz:* „Stahl und Eisen", 1933, p. 551.

[18] *Schmuckler:* „Stahlbau", 1931, pp. 133 et seq. also *Klöppel:* „Stahlbau" 1933, pp. 14 et seq.

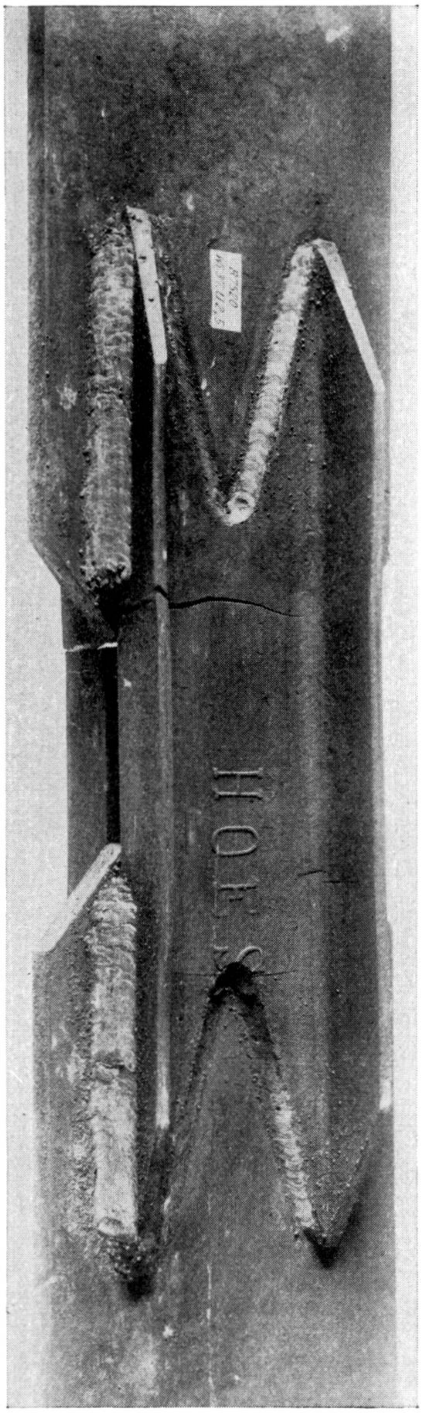

Fig. 7.

Fig. 8.

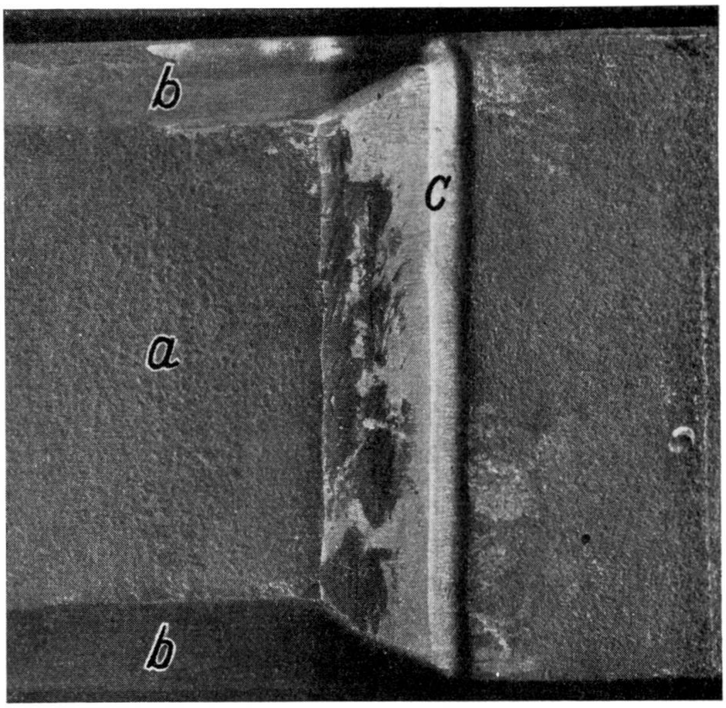

Fig. 9.

because a pronounced stress-peak occurred in the tension zone of the web. The T member in Fig. 11 was the outer stiffening of the web.

In investigations of this kind, two part-problems have to be dealt with: (1) finding the best type and shape of connections; and (2) deciding as to the extent of the moment which has to be taken up.

As regards the type of joints, it should be noted, *inter alia,* that it is probably best to attach the flanges of cross girders so, that the joint acts in tension, the simplest way being to secure them to the web of the main girder by a butt-weld. The fixing of the web of the cross girders is done best by simple fillet welds, the thickness required is being investigated at present. If the cross girders rest directly on the tension flange of the main girder, and are

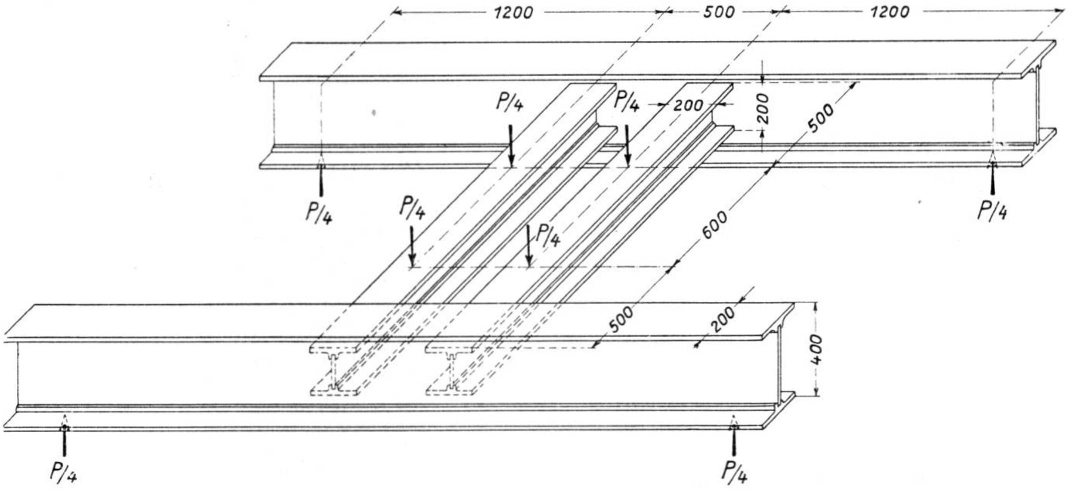

Fig. 10.
Tests with welded systems

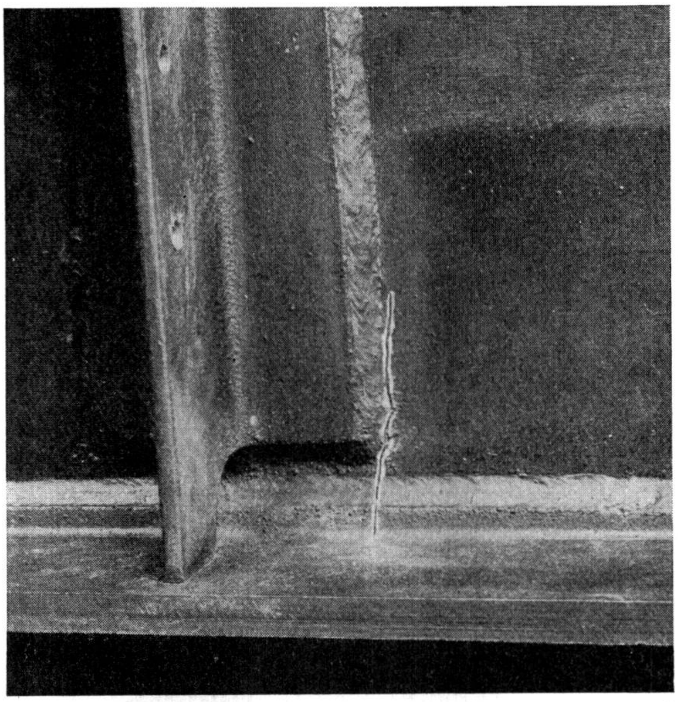

Fig. 11.

welded to the main girder flange, a stress peak will occur in the tension flange of the main girder at the place where the cross is attached by welding, such a stress-peak will considerably reduce the strength under frequently recurring loads. This reduction in the strength of the tension zone of the main girder will often have to be put up with, if good joints are to be made for the cross girders. The stress in the main girder should be selected accordingly.

d) The fourth question concerns the extent of the *stresses occurring with welding*. It is not intended to discuss here how these stresses develop, but merely to deal with the results available. It is known, apart from other points, that, in butt-welds and fillet welds, high stresses occur locally which may reach the yield point[19]. In butt-seams welded in one operation, these stresses take the form of compressive stresses at the edge at right angles to the seam, and tensile stresses in the middle portion[20]. The highest stresses are strictly limited in locality, for which reason only very small permanent deformations are necessary for considerably reducing local stresses[21]. From many fatigue tests it may also be estimated that the self-stresses set up by welding have at all events no marked influence on the strength of the connection, because the parts involved are designed as tension members, and their ends were free when they were welded. The self-stresses in the welds may also be influenced by the method of welding and by the material[22]. The following phenomenon is probably more important than the welding stresses.

If a tension member is made with a butt-seam, contractions take place on the weld parallel to the tensile force in the member, partly because the molten weld metal in the joint shrinks when it solidifies and cools. Shrinkage increases with the width of the joint and, hence, with the angle of opening also[23]. If the tension members are clamped at the ends, this more or less prevents the weld from shrinking; in addition to which changes in volume are set up by the heating and cooling, which become apparent in the material adjoining the weld. This sets up other stresses besides those peculiar to the weld itself. These external (or shrinkage) stresses are highest when the bars are very short in length, and when the clamping does not "give". The amount of shrinkage has therefore to be absorbed by a short length of bar. If the bar is long and its clamping more or less yielding, the influence of shrinkage is diminished due to the greater expansion of the long bar and to the deformation of the structural parts which have to hold the member. If the member is, say, 3 m long, and if the points where the bar is restrained do not "give" for a stress in the member of $\sigma = 1050$ kg/cm^2, a change in length of approx. 1.5 mm takes place for this stress. According to what is known, this should be sufficient to equalize the shrinkage of a weld cross-sectional area of roughly 100 mm^2. If the free length of the member were only 0.5 m and the support did not "give", the shrinkage stress would have to produce stresses going above the yield point if 1.5 mm of shrinkage is be equalized.

The shrinkage forces calculated in this way are actually smaller, because the deformations develop around and in the weld while the material is cooling. However, the examples show that special attention should be given to the order in which the welds are made when building up welded structures.

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19 *Graf:* „Stahlbau", 1932 pp. 181 and 182; also 1933, pp. 93 and 94; „Zeitschrift des Vereins deutscher Ingenieure", 1934, p. 1426.

20 cf. inter alia *Bierett:* „Zeitschrift des Vereines deutscher Ingenieure", 1934, pp. 709 et seq.

21 *Bollenrath:* „Stahl und Eisen". 1934, p. 877.

22 Added to which there is also the sensitiveness of the weld to cracking. cf. *Bollenrath* and *Cornelius:* „Stahl und Eisen", 1936, pp. 565 et seq.

23 cf. inter alia *Lottmann:* „Zeitschrift des Vereines deutscher Ingenieure", 1930, pp. 1340 et seq., *B. Malisius:* „Elektroschweißung", 1936, pp. 1 et seq.

It should also be noted that the shrinkage forces occurring while the weld is being made may set up very high stresses in the weld-seam, where only part of the seam has been welded and this part can cool down. Although the shrinkage force is not yet high in such cases, the elasticity of the structural steel is very small in extent, so that shrinkage must be almost entirely taken up by the partially applied weld. This means that the weld should be made in a single operation, before cooling actually sets in properly and the first part of the weld must be particularly ductile.

To get some idea of the actual shrinkage forces, the writer carried out experiments in 1934 on this point. A frame as in Fig. 12 has stout cross-pieces

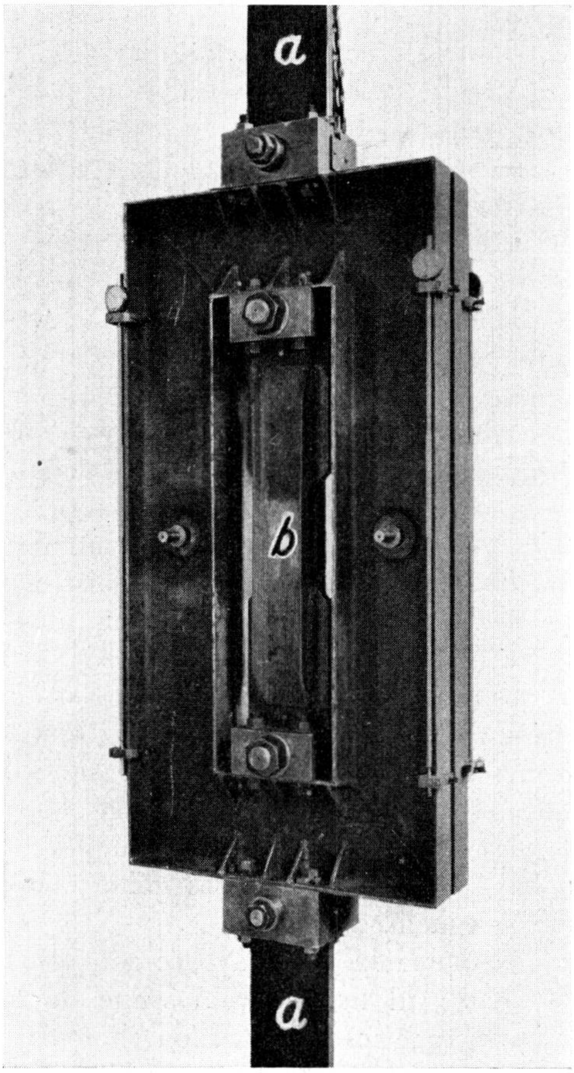

Fig. 12.

at the ends with one hole in each. By means of bolts (one of them tapered) passing through the holes, stout pieces of flat iron a — a were secured in the frame so as to be practically immovable. The inside ends of the iron strips were joined by welds, in the case of Fig. 12 with [-irons b by side-fillet welds. The tests made up to now under these conditions have given, for butt-welds, shrinkage stresses of approx. 250 kg/cm². Further details of these tests will be given in a separate report.

Summary.

The use of welding for bridge and structural engineering work in Germany has been accompanied by numerous investigations which enable the type of joints and their design to be selected for any particular conditions. Very extensive research was undertaken with a view to ascertaining the carrying capacity of welded joints and structural elements under frequently recurring loads such as occur in bridges, cranes, etc.[24] Numerous tests have also been made for determining the stresses which remain in the joint itself and in the structure after welding.

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[24] cf. among the papers cited, inter alia, „Dauerfestigkeitsversuche mit Schweißverbindungen", published by the VDI-Verlag in 1935.

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