The reduction of thermal stresses in welded steelwork

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The Reduction of Thermal Stresses in Welded Steelwork.

Verminderung der Wärmespannungen in geschweißten Stahlbauten.

La réduction des contraintes thermiques dans les constructions métalliques soudées.

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In welded steelwork thermal stresses are a combination of the rolling stresses which are present in the individual members as the result of rolling, and of the welding stresses which are caused by the welding operation. These two kinds of stress are superimposed on one another, and two problems consequently arise:

1) That of minimising thermal stresses in welded steelwork, and —

2) That of ascertaining whether the thermal stresses which are unavoidably present in welded steelwork impair its load carrying capacity.

Rolling stresses are the result of uneven cooling, and they occur even in the simplest shape of rolled section, which is the round bar. In such a bar the core is in tension and the outside is under a corresponding amount of compression, because after being rolled the bar cools from the outside inwards; hence the plastic core follows the earlier shrinking movement of the outside without any stresses arising therein, but when the outside has solidified it is not, in its turn, able to follow the subsequent shrinking of the warm and plastic core, and as a result it is placed in a state of elastic compression which is balanced by elastic tension in the core after the cooling process is completed. These conditions of stress may be demonstrated by means of an experiment as follows:

If a round rolled bar 80 mm in diameter by 1000 mm long is turned down to 30 mm diameter, thereby releasing the core which is in tension from the surrounding outside portion which hinders the contraction of the core, it will be found that the core shortens by about 0.15 mm. This would correspond to an average rolling stress of approximately 300 kg/cm^2 in the core.

Rolling stresses can be reduced by annealing, but not eliminated, because the same conditions remain after cooling as were present in the red hot bar during the rolling process. The slower, however, the hot bar is allowed to cool, the smaller will be the remaining stresses. The following experiment serves to show that a bar cannot be entirely freed from stress even by repeated annealing operations:

A round rolled bar of 70 mm diameter by 1000 mm in length was made red hot 63 times and on each occasion was allowed to cool slowly. In this way it became about 26 mm shorter because, when the bar was heated, the outside (which as already explained is under a state of elastic compression) became hot and plastic before the core; the tensile stresses which were present in the core while still cold and elastic served to compress the already plastic outside portion, and in this way the bar was made shorter and correspondingly thicker. After this action had taken place the glowing bar was free from stress, but when it had cooled down as a whole the thermal stresses, as already explained, became the same in magnitude and distribution as after the original rolling. If the experiment were repeated often enough the bar would finally become a ball.

In less simple cross sectional shapes of rolled section the rolling stresses are generally higher than in the simple round bar. For instance, in the webs of I-beams NP 50 the compressive rolling stress has been found to be 170 kg/cm², and in the webs of broad flange beams $42^1/_2$ cm deep compressions up to 1600 kg/cm² have been measured.[1] The large difference is accounted for by the differing proportions of the flange to the web. In the welding of steel structures it is advantageous to use the simplest possible cross sections with the smallest possible rolling stresses, and for this reason, for instance, slit I-beams are not suitable.

The *welding stresses* are superimposed on the rolling stresses in the process of welding; their magnitude and extent depends on the appliances used and on the sequence followed in the welding operation.[2, 3] Under otherwise equal conditions, such stresses will increase with the size of the cross section of the seam, and the latter should, therefore, not be made larger than necessary. For the same reason X-seams are preferable to V-seams, for with the same angle of 90^0 and the same thickness of material a given load capacity can be carried by an X-seam of only half the cross section of a V-seam — involving, therefore, only half the welding work — and correspondingly smaller welding stresses will result. Apart from this the eccentric position of the V-seam causes the plates to be welded to be thrown out of position which can only in very rare cases be compensated by appropriate arrangement of the pieces to be welded, so that adjustments have to be made after the welding is completed, at great expense and to the detriment of the structure.

In seams of equal cross section the welding stresses are heavier when the welding is done with a thick electrode in a single layer than when it is carried out using thin electrodes depositing several layers. In structural steelwork no electrode larger than 7 mm diameter should be used, but on the other hand it is not advisable to go below 4 mm diameter because the heavy sections usual in this class of work cannot then be adequately fused to ensure perfect penetration.

The welding stresses can possibly be somewhat reduced by making use of intermittent welding, but the numerous beginnings of runs, with their attendant disadvantages, must then be taken into account. On the whole, therefore, it is better to carry out the welding in a single run, beginning at the middle of the work and proceeding simultaneously towards either end.

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[1] *Dörnen:* Schrumpfspannungen an geschweißten Stahlbauten. Der Stahlbau 1933, N° 3.

[2] *Schroeder:* Zustandsänderungen und Spannungen während der Schweißung des Stahlbaues für das Reiterstellwerk in Stendal. Der Bauingenieur 1932, Nos 19/20.

[3] *Krabbe:* Entstehung, Wesen und Bedeutung der Wärmeschrumpfspannungen. Elektroschweißung 1933, N° 5.

Bierett, in an article printed in Stahlbau, 1936, Nº 9,[4] and also in his paper for the Congress, distinguishes between natural and secondary welding stresses, and further subdivides the latter into those which are the result of internal and those caused by external conditions. The natural welding stresses are on a par with rolling stresses and must be dealt with in the same way as the latter, being usually no greater in magnitude[5] and, in *Bierett*'s opinion, not dangerous. The secondary welding stresses due to internal causes are the result of building up the seam over its whole length and throughout its thickness in separate layers one over another, and to welding still going on while previous deposits are already cool. In the bottom layers of thick seams these stresses are especially critical and may easily give rise to cracks whereby the seam is rendered unsound at its core; they can, however, be reduced to insignificant amounts by careful hammering of the cold bottom layers and — as *Bierett* recommends — by suitable heat treatment, allowing the portion of the seam which has already been deposited to remain hot until the whole of the seam is completed, at any rate as regards the bottom layers. In the case of particularly important seams — as, for instance, the butt joints in tension flange plates — it is advisable to make the seam thicker than the flange plate itself, and to make the joint red hot for a distance of about one quarter of a metre on either side, while hammering the projecting portion of the seam down flush with the plate. In doing this, working at a blue heat should be avoided. In this way not only can all secondary welding stresses due to internal effects be eliminated, but in addition the grain structure of the weld seam is rendered more dense, and the contact between the weld metal and the parent metal is made more intimate at the delicate place of junction. It is thus possible to secure almost the same conditions of stress in the joints of flange plates, and also in those of web plates (and even in universal joints in plate web girdes) as if the plates had been rolled in single lengths. For this purpose good results have been obtained using gas burners which consist of long pipes having rows of burner holes suitably arranged over the work, the necessary gas being taken from the gas pipes in the shop. For use on site the fuel can, if necessary, be compressed gas, or liquefied gas taken from cylinders.

Welding stresses due to external conditions occur when the parts to be welded are unable to follow up the shrinkage of the seams. As regards most types of member now in use — such as solid webbed forms of plate girder, plated arches, plated frames and plate webbed beams reinforced with arches and Vierendeel girders — these stresses again can, to a considerable extent, be eliminated by taking suitable precautions. This will be illustrated in the *first* of the examples below.

The plate web girder, in its simplest form, best adapted to welding, consists of a web plate and two flange plates which extend over the whole length of the girder without any joints. In this form only the neck seams connecting the flanges to the web have to be run and the stiffeners welded into place. In the neck seams the secondary thermal stresses can, if it is considered necessary, be eliminated by heating every seam over the whole length at once to approximately

[4] *Bierett:* Welche Wege weisen die Erkenntnisse über Schrumpfwirkungen den Arbeitsverfahren für die Herstellung von Stumpfnähten im großen Stahlbau? Der Stahlbau 1936, Nº 9.

[5] *Dörnen:* Schrumpfungen an geschweißten Stahlbauten. Der Stahlbau 1933, Nº 3.

400—500° C. If care is taken that the flange plates are able to follow the transverse shrinkage of the neck seams without any resistance being offered, no secondary thermal stresses due to external stressing need arise. The stiffeners, as a rule, are best welded into place after the web plates and flange plates have been welded to one another, because after the neck seam are welded the web is at first mainly in compression, and by welding on the stiffeners parallel or at right angles to the axis of the girder such compression is diminished or is converted into tension, while the tension present in the flanges is likewise reduced. At those places where vertical stiffeners are to be welded on a gap of about 20 cm length of the neck seam should be left open on either side, this gap to be filled in after the stiffeners have been attached. This procedure serves the better to equalise the shrinkages in the web plate produced by the vertical stiffener seams.

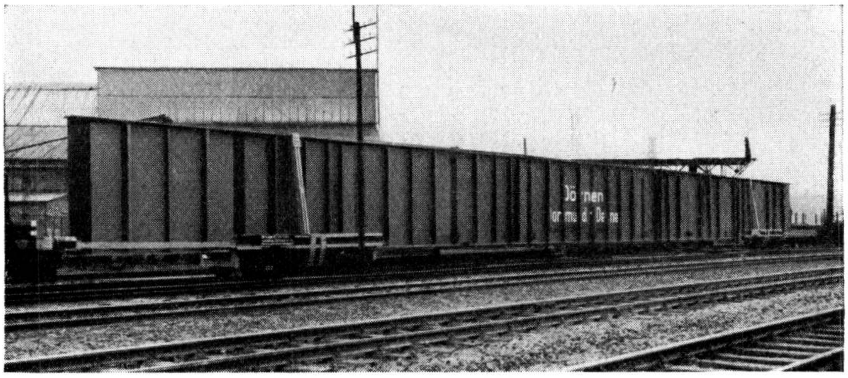

Fig. 1.

For the vertical seams 3 mm thickness is enough. This simple arrangement is limited to cases where the dimensions of the girder do not necessitate an excessive thickness or length of flanges, but too great thickness of undivided flange plate is not desirable, as it leads to difficulties in rolling. Using steel St. 37. 12 it is not advisable to exceed 60 (70) mm thickness. Where the thickness is considerable the flanges are preferably divided into several layers each of which is without joints if possible. In such a case the web plate should first of all be welded to the innermost flange plate the reason for this being that the tensile forces in the neck seams, shrunk longitudinally, are smaller for equal seam sections, if the parts to be welded are comparatively light and flexible. The stresses are further diminished by the use of fillet seams for attaching the additional flange plates. The stiffeners may most conveniently be attached after the web plate has been welded to the first, innermost, flange plate.

If the girders are too long to be treated in this way the web plate or the flange plates of both must be fabricated with joints. Generally speaking it is more often the web plate and less so the flange plates which are jointed. The web plate is first of all separately completed (using butt joints) and if necessary these seams are relieved of secondary thermal stresses caused by internal effects by heating and are examined by X-rays. Thermal stresses due to external effects may be avoided by clamping the pieces to one another while they are being welded, this work should not be left to the shrinking weld to do. The quality of any butt weld depends primarily on ensuring perfect formation of the root layers, which must be free from even the finest cracks. Next, the edges of the web plate are cut to

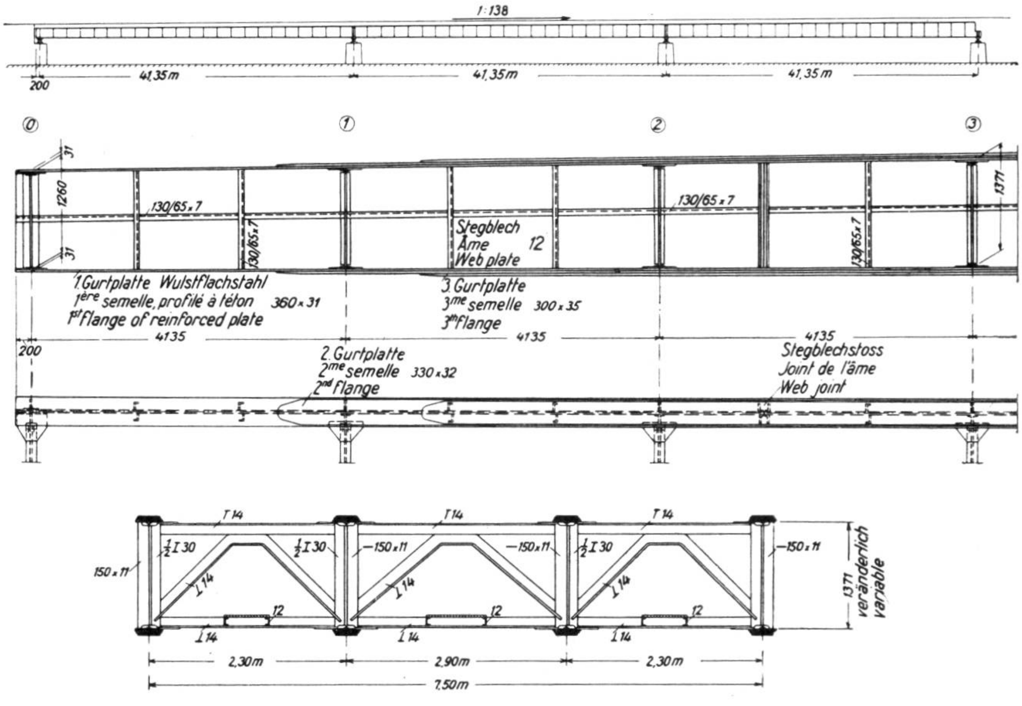

Fig. 2.

Welded river spans.

fit the camber of the girder, then preparations are made for welding on the flange plates. The stiffeners are welded into place after the web plate has been welded to

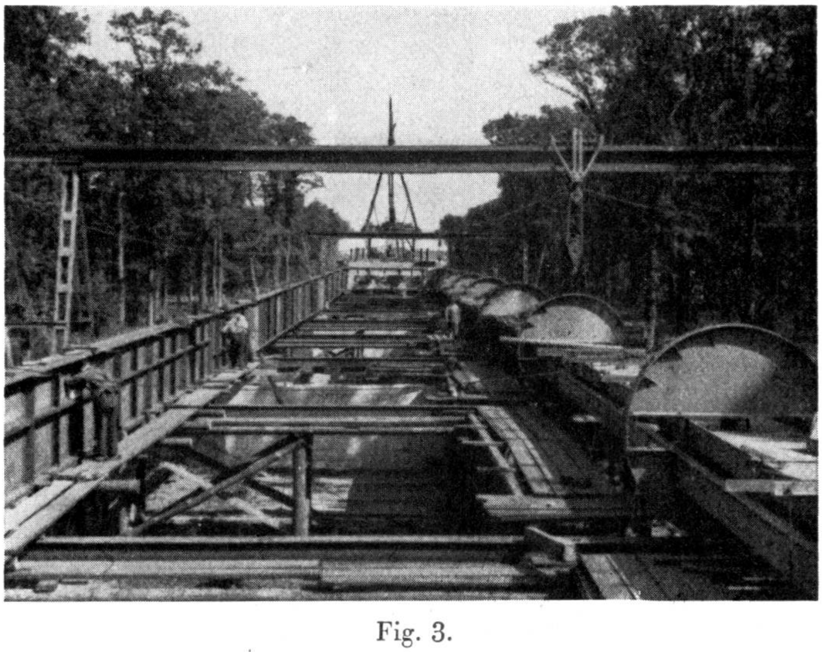

Fig. 3.

the innermost flange plates. Vertical stiffeners close to vertical butt joints in the web plate are to be avoided so that the latter may not receive too heavy tension.

If, further, the necessity for joints in the flange plates cannot be avoided, these are welded before making the connection to the web plate, the result then being as good as if there were no joints. In the case of the flange joints secon-

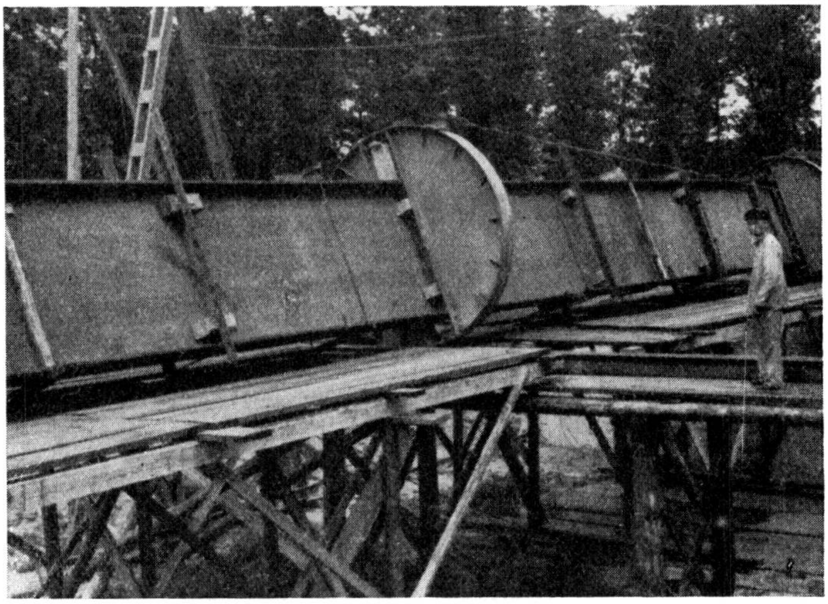

Fig. 4.

dary thermal stresses due to internal effects should be avoided by suitable heating. Secondary thermal stresses due to external effects do not arise. The joints in the flange plates should preferably be placed where the stresses are

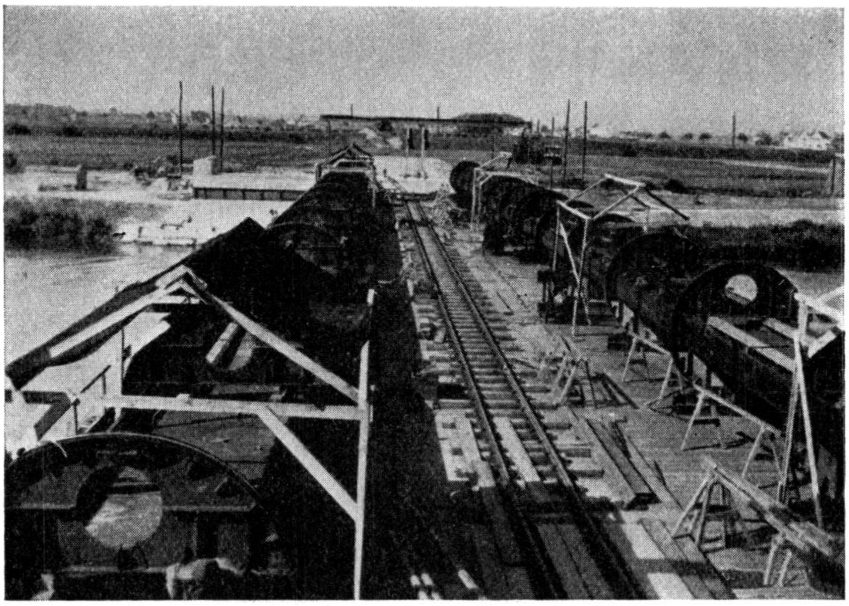

Fig. 5.

least; for instance, in the case of continuous girders they should be placed where the bending moment is at a minimum so that the material is under only a light stress. It is desirable that the flange plates and web plates should not be jointed at the same cross section of the girder.

In this way it is possible to build up very large plate web girders, in very long lengths. Fig. 1 shows such a girder 63 m long by approximately 4 m deep, weighing about 105 tonnes, which was so constructed in the shops that the whole could be carried on special railway rolling stock belonging to the works, and was transported in this way from Dortmund-Derne to the site at the Rügendamm. For still greater dimensions, the web plate and the flange plate may be entirely welded together at the site, the same arrangements being provided as in the workshop. Of course all the seams should be arranged so that they can be readily

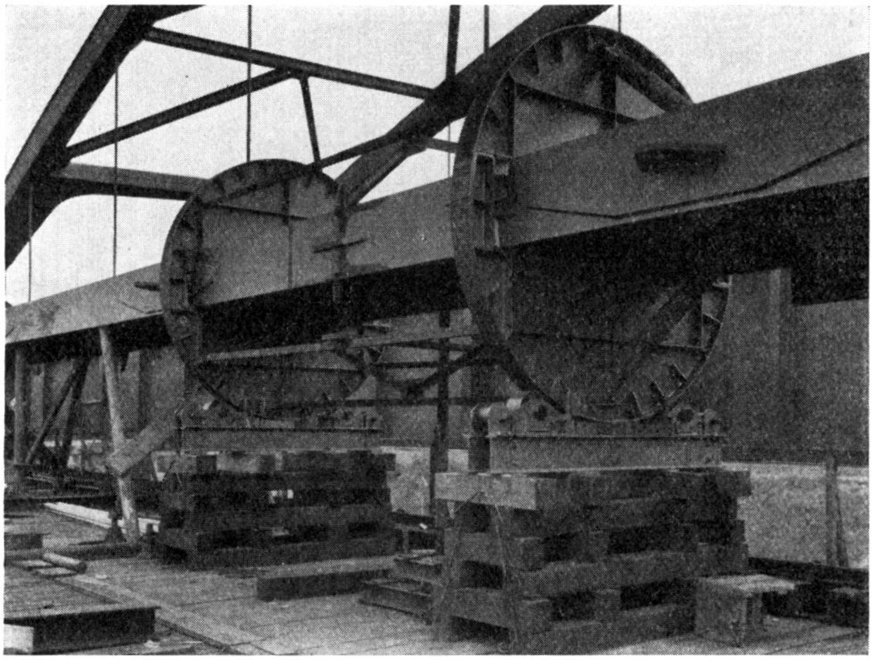

Fig. 6.

welded from above, the work being held in a suitable turning device and protected from the weather. As an example the construction of four plate web girders each 130 m long may be mentioned, these girders resting finally on four supports as shown in Figs. 2, 3 and 4, for the Elbe bridge at Dömitz, using St. 52. The girders were constructed exactly as described above and no difficulties arose, although this was the first occasion on which this method had been applied. In the same way the stiffening girders, approximately 95 m long, for a *Langer* girder bridge with reinforcing arch carrying the Reichsautobahn over the Lech near Augsburg were built (Figs. 5 and 6).

So far, it will be seen, the conditions for welding plate girders are particularly favourable, but they become more difficult when it is impossible to avoid the necessity for universal joints and when it becomes necessary to weld together finished sections of the girders. When this is the case it is impossible to avoid thermal stresses due to external effects, but by adopting a suitable sequence in the operations it is possible to keep the stresses within reasonable limits and to control them in such a way that, for instance, within the region of maximum tensile stresses due to welding a compressive stress is pre-imposed, and vice-versa. An example of this is shown in Fig. 7 which represents the joint for a plate web girder for the bridge over the Strelasund in the Rügendamm crossing, formed of welded continuous girders resting on six supports and

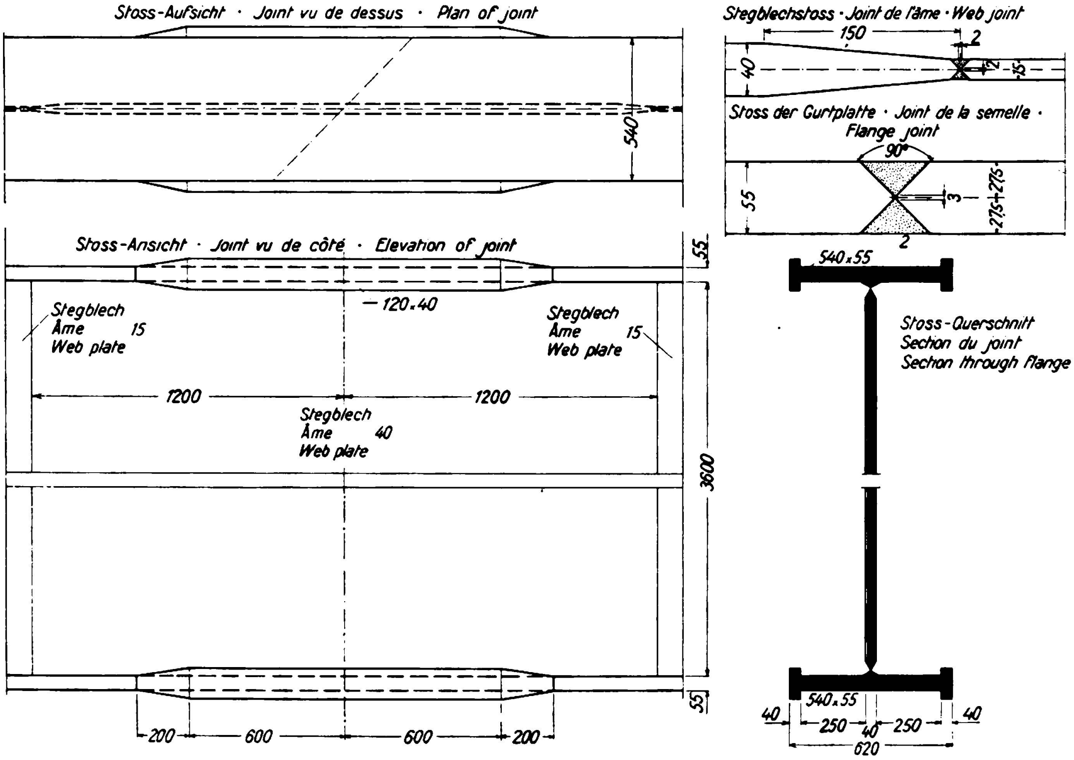

Fig. 7.

spanning over 5 openings of 54 m each. After accurate measurements carried out by the Staatliches Materialprüfungsamt in Dahlem a welding procedure was laid down which called for pre-stressing to approximately 300 kg compression in order to reduce the maximum tensile stresses due to dead and live load at the

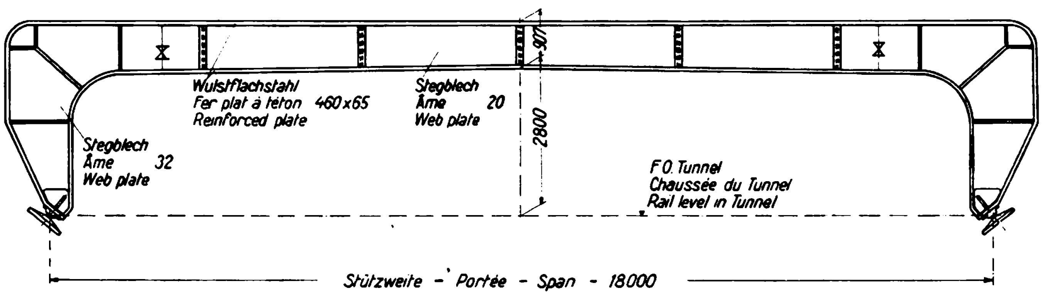

Fig. 8.

most heavily stressed portion of the tension flange. The reinforced web plate was required in the middle third of the girder, and had to be given a pre-stress of about 350 kg tension.

In plate webbed arches and frames, the conditions in regard to thermal stresses are hardly less favourable than in the case of plate webbed girders, provided that care can be taken to avoid stresses due to external effects. In the case of the plate webbed frames shown in Fig. 8 (Duisburg) the three portions of the web plate were first welded to one another and were then connected to the jointless flanges bent to the required curvature.

Thermal stresses due to external stressing were avoided by adopting the procedure of welding from the middle of the span outwards towards both hinges, and also by the adoption of suitable arrangements for attaching the flange plates to the web plate.

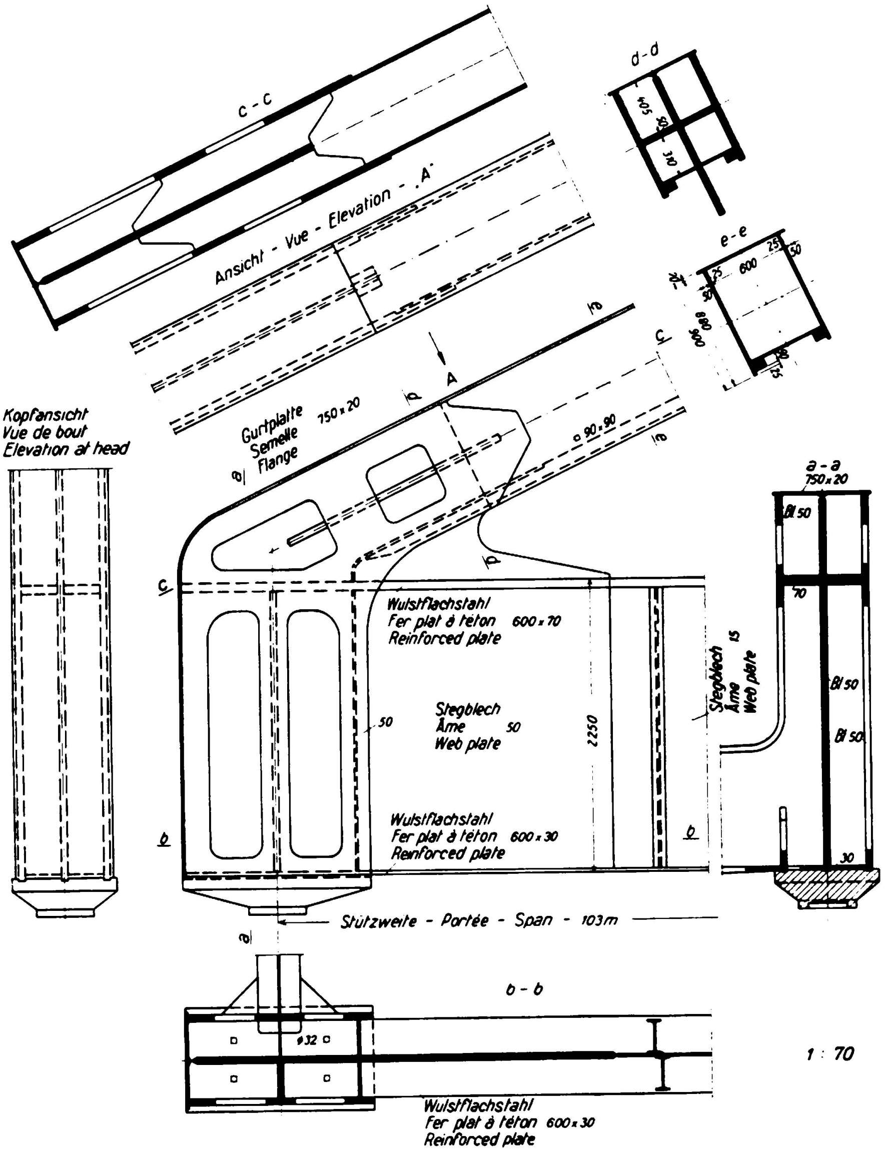

Fig. 9.

So far reference has been confined to structures which may be regarded as plane. The shrinkage stresses due to external effects are more difficult to cope with in three-dimensional structures, and in these special precautions may have to be adopted. Thus Fig. 9 shows the end of a welded *Langer*-girder of 104 m span, composed entirely of rolled sections welded together. The welding of this job, while not attended by any insuperable difficulties, was nevertheless not easy, and moreover a great deal of welding work was involved. The end piece was repeated eight times in all, for use in two bridges. Exhaustive investigations, involving the use of X-rays, indicated perfect results. The experience gained from this work led to the design of the end features as shown in Fig. 10 for use in *Langer*-girders of 95 m span, and in order to simplify the welding work and to reduce the shrinkage stresses a forging was inserted between the double walled arch and the single walled stiffening girder. This piece has a thick downward rib which passing through the slotted flange plate, is welded to the web plate of the girder, thus providing a very simple means of welding without the risk

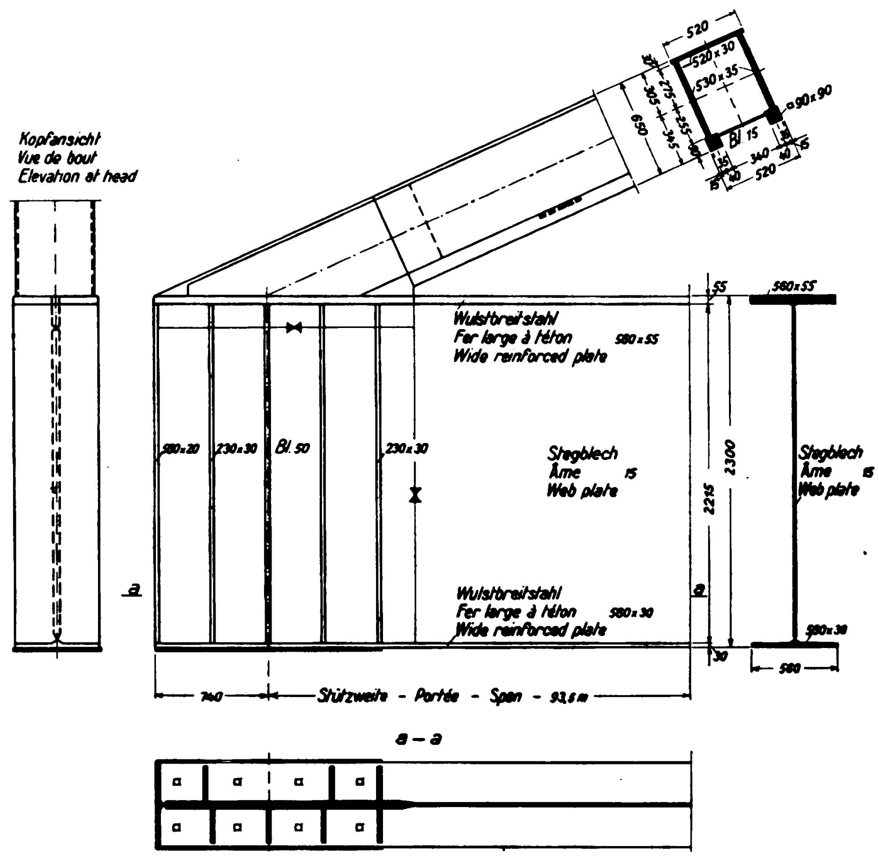

Fig. 10.

of giving rise to shrinkage stresses due to external conditions. The double walled polygonal arch is carried on the forging. The arrangement will readily be understood and requires no further explanation.

In order to avoid shrinkage stresses there should be no hesitation in special cases to introduce riveting in an otherwise welded structure, and in this way an economy may often be realised. Welding should not be adopted always and in all circumstances. The author has already argued this point and found agreement before the previous International Congress at Paris in 1932. For instance, it is often justifiable to make the connections between the longitudinal and the cross girders, and those of the wind bracing, by the use of riveting even if the rest of a bridge is welded. In the erection of large bridges it is very often necessary to simplify the work by the use of bolted connections at such places, to steady the erection work of further superstructures. It may often be advantageous to arrange these bolted connections in such a way that rivets may be substituted and welding thereby saved. In the author's experience the most economical way to construct truss bridges is to form the individual members of the trusses and of the floor from rolled sections which are individually welded but to make the intersections and connections by riveting. In the case of the tension members the weakening caused by the presence of the rivet holes can be compensated by local reinforcements welded on. The author has been responsible for several structures on these lines which have been economically very successfull; one of these is represented in Fig. 11.

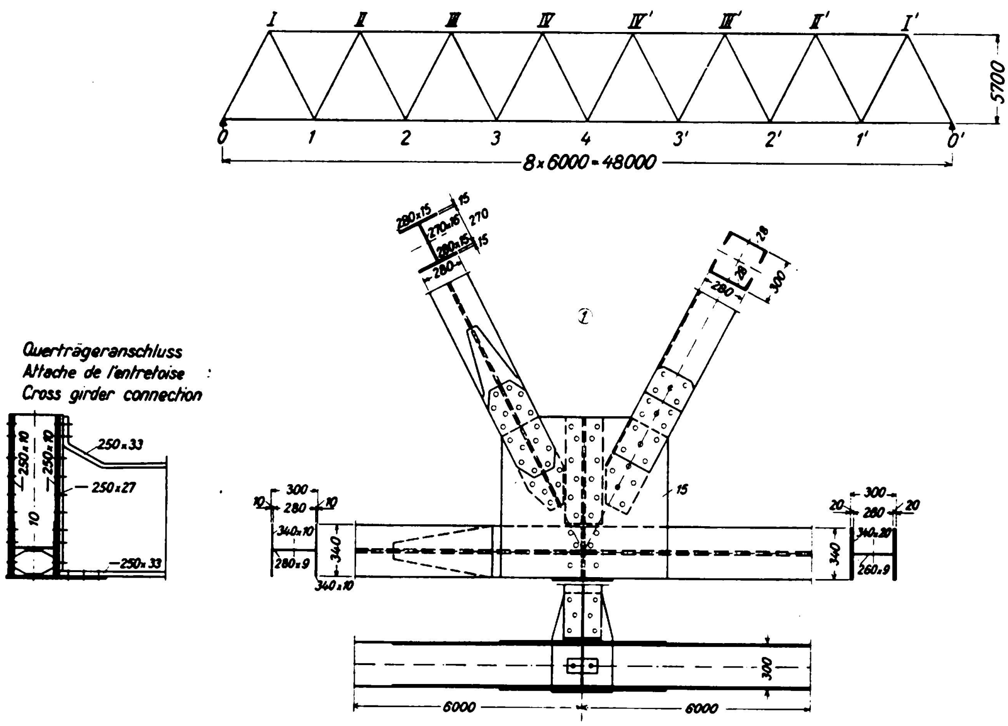

Fig. 11. Main girder. Intersection No. 1.

The *second* of the two problems is answered if the thermal stresses (using also normal sized welds) can be shown to be no greater than the rolling stresses in rolled joists, for the latter are shown by decades of experience not to be dangerous. In several forms of construction with specially thick seams, however, welding stresses have been measured which may sometimes exceed 2000 kg/cm^2, and stresses of this magnitude cannot *a priori* be assumed to be free from danger. On this matter *Dr. Schröder* of the Reichsbahn gave an explanation as early as 1931, and he also gave the reason why the welding stresses are so heavy. It has, however, been found by means of experiments on small test pieces that these welding stresses are not superimposed upon the stresses due to the loading, and therefore do not impair the carrying capacity of the member in question. No perfectly satisfactory explanation of this result has been advanced, but the fact remains. Since apprehension continued to be felt an the point, *Dr. Schaper* decided to carry out experiments on pieces of full size, and these are about to be published.[6] One of these experiments may be described here: The two corners

[6] Meanwhile published: *Schaper*: Die Schweißung in Ingenieurhochbau und Brückenbau. Elektroschweißung 1937, N° 7.

of an all-welded frame forming part of the superstructure for a passenger subway in Duisburg (Fig. 8) were cut off and were subjected to loads in a 600 tonne press at Dahlem. The welding stresses represented in Fig. 12 were measured in these two pieces. The corner carried an experimental stress of over 2500 kg/cm² without showing visible damage. No heavier load was possible in this machine, but the corner piece is now to be tested in another. This test proves that welding stresses of over 2000 kg/cm² do not impair the carrying capacity of welded structures. The same result was found in other large scale experiments by *Schaper*.

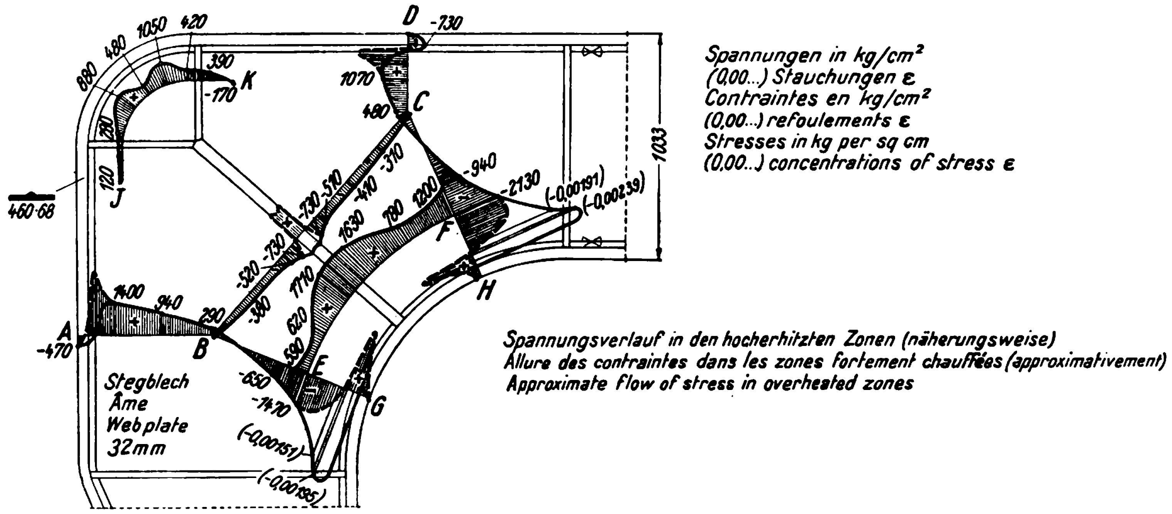

Fig. 12.

Measurement of welding stresses during construction.

Finally reference may be made to an article by *Körber* and *Mehovar*[7] in which it is shown that the mechanical properties of rails fresh from the rolling mill are altered by storage alone. The elongation and the reduction of area before breakage (which are criteria and measurements for the soundness of the material) undergo considerable increase by longer storage. This effect may be more rapidly made and more marked by tempering and annealing, a fact which is attributed to the equalisation of stresses in the texture of the material. These conditions, which are true of rail steel, cannot of course be applied straight away to structural steels, but at the same time the behaviour of the latter cannot be altogether different and it may be inferred that in the case of weld seams, also, storage may be attented by some compensation of the internal stresses. Since, in the articles cited, a further improvement in the elongation test and in the reduction of area test was found to attend annealing which caused recrystallisation, the possibility suggests itself that important weld seams and the adjacent material might be treated in the same way. The author is of opinion that a further improvement in the quality of weld seams is conceivable from this point of view.

[7] *Friedrich Körber* und *Johannes Mehovar:* Beitrag zur Kenntnis der zeitlichen Änderungen der mechanischen Eigenschaften walzneuer Schienen insbesondere aus Thomas-Stahl. Mitteilungen aus dem Kaiser-Wilhelm-Institut für Eisenforschung zu Düsseldorf. Band XVII, Lieferung 7, Abhandlung 277.