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III

Practical questions in connection with welded steel structures.

Praktische Fragen bei geschweißten Stahlkonstruktionen.

Questions pratiques concernant les constructions soudées.

III

General Report.

Generalreferat.

Rapport Général.

Geheimrat Dr. Ing. G. Schaper,
Ministerialrat, Reichsverkehrsministerium Berlin.

We have before us a problem of great and immediate importance, namely the application of welding to structural steelwork and bridge engineering. At the Congress of 1932 in Paris this question was still viewed with scepticism and such applications of welding were made the subject of many warnings, fears being expressed as to the magnitude of the shrinkage stresses, the cast metal structure of the weld seams, and the difficulties associated with the execution of welded work. The papers now presented show, however, that since 1932 welding has been more and more widely adopted in building and in bridge engineering and that its progress has never been interrupted. All objections to the use of welding in structures to carry stationary (or mainly stationary) loads must be regarded as completely overcome, whether in building or in road bridges. For this purpose solid webbed designs have been the type most used in welded work, but examples of welded frame structures are not lacking.

In the construction of railway bridges there is still, in many countries, some hesitation and backwardness in the adoption of welding; in others — especially Germany — great progress has been attained in this direction, though up to the present only solid webbed railway bridges have been welded. Since 1932 exhaustive and extensive experiments have been carried out, especially in Switzerland and in Germany, with a view to studying the fatigue resistance of welded connections and of all-welded plate web girders, and these have shown that certain forms of welded connections — particularly butt welds — possess excellent fatigue resistance so long as the parent material has good welding characteristics, the mechanical properties of the electrodes are suited to those of the parent material, the roots of butt seams are scraped out and rewelded, and the transition from the seam to the parent material is not too abrupt at the surface and is free from notches (milling being here of great value).

The experiments have also shown that the ends of flange plates which are to be added to plate web girders should be tapered longitudinally and should be connected by transverse fillet welds merging smoothly into the plate below, and that longitudinal fillet welds at the ends of such plates must be adapted to the diminishing thickness of the plate in question and be suitably worked over.

Experiments and experience have shown that apprehension as to the magnitude of shrinkage stresses is groundless: in the seams connecting the web to the flange plates such stresses may, indeed, be very high (2000 kg/cm^2 or more) but in the girders themselves they do not reach this value and are, in fact, no greater than the rolling stresses present in rolled joists, especially in broad flange beams. Such members carry almost as heavy loads as if they were free from rolling stresses, and the same argument obviously applies to welded girders. Even the high shrinkage stresses in the "neck" welds connecting the web to the flange are attended by no danger; this has been proved by exhaustive experiments in reference to the fatigue bending resistance of welded plate web girders which, despite the heaviness of the shrinkage stresses, are found to possess greater fatigue bending resistance than riveted girders, and the explanation lies in the fact that the heavily stressed seams are surrounded by less heavily stressed portions which relieve the high stresses in the part adjacent to them as soon as the latter exceed the yield point.

In other fields of engineering important constructions subject to heavy dynamic stresses are successfully being welded, such as pressure plant for use in shipbuilding, dredger buckets, railway rolling stock, locomotives, etc. In Germany, as in other countries, small and large ships are being built in St. 37 and St. 32 entirely by welding, and the performance obtained with such ships is excellent, despite the fact that in a heavy sea very powerful alternating stresses of high frequency arise, especially in small ships.

Reference may now be made to certain notable welded bridges in different countries.

In *Hungary* the Raba Bridge at Györ was built by welding (Fig. 1) this being a road bridge of 63 m span, in which the main girders are trapezoidal trusses.

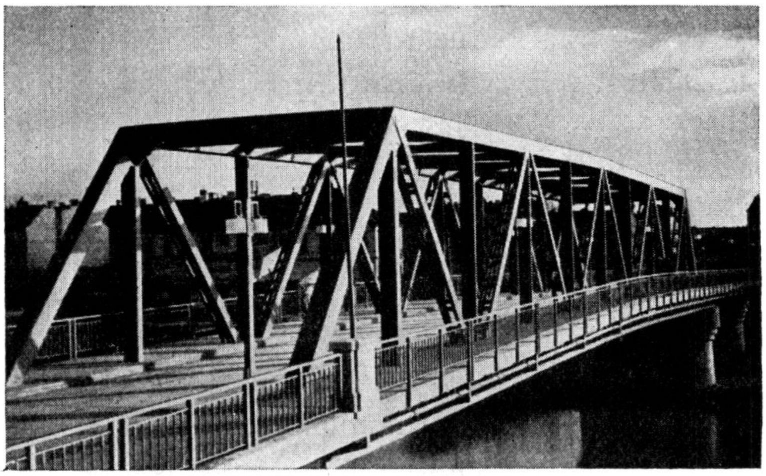

Fig. 1.

In *Poland* there is the well known road bridge near Lowicz (Fig. 2) constructed as early as 1928, for which we are indebted to Prof. *Bryla* of the Technical University of Warsaw. Here the main girders are trusses of 27 m span.

In *Belgium*, in addition to a number of plate-webbed girders, special use has been made of welding in the construction of Vierendeel girders. Fig. 3 shows a road bridge of this type covering 61 m span in the main opening, with plate web girders over the side openings.

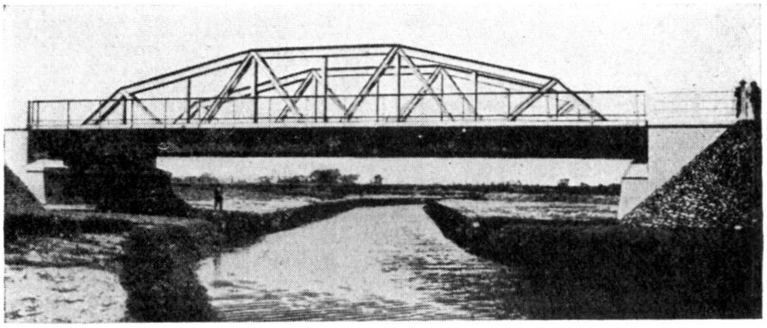

Fig. 2.

In *France* a notable all-welded plate web girder railway bridge near Saint-Denis deserves special mention (Fig. 4). This bold piece of work has been carried out in high tensile steel St. 54, by Messrs. *Cambournac*. The main girders are continuous over three supports and the depth of the web plates is 2.30 m. One of

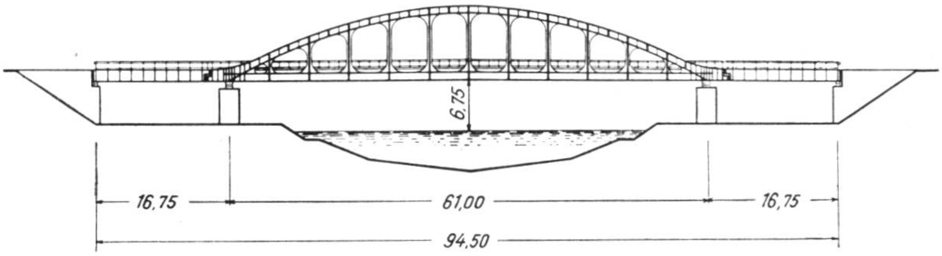

Fig. 3.

the two main girders covers spans of 26.9 and 35.2 m, the other 28.9 and 32.3 m.

In *Yugoslavia* the railway bridge at Zagreb (Fig. 5), which is a plate-webbed frame bridge over three spans, is entirely of welded construction.

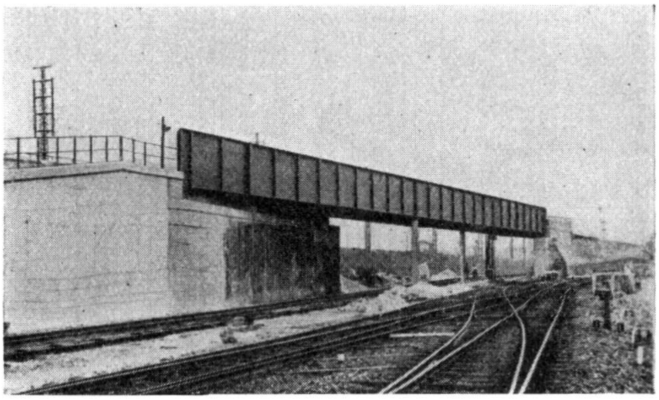

Fig. 4.

In *Roumania* there is a road bridge with trussed girders covering 30 m span (Fig. 6).

In *Sweden* there is the well known Mälarsee bridge near Stockholm (Fig. 7) in which welding has been used for the whole of the roadway, the upper wind bracing, and the supporting columns on which the roadway is carried, amounting to a total weight of 2000 tonnes.

The Traneberg Bridge (Fig. 8), which is otherwise of reinforced concrete construction, has welded roadway girders and upper bracing, the weight of the

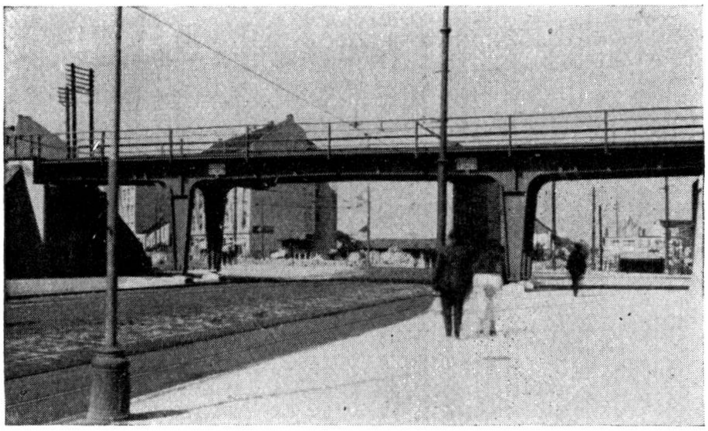

Fig. 5.

welded portions being 1300 tonnes. Another Swedish bridge which is entirely of welded construction is that at Palsund near Stockholm (Fig. 9), which spans over the water by an arch girder of 56 m span and over the approaches by beam bridges of 12 m span carried on steel columns. The overall length of this bridge is 276 m and the total weight 1100 tonnes; the cross girders are made of St. 52 and all the remaining portions of St. 44. Fig. 10 is a view of the completed bridge.

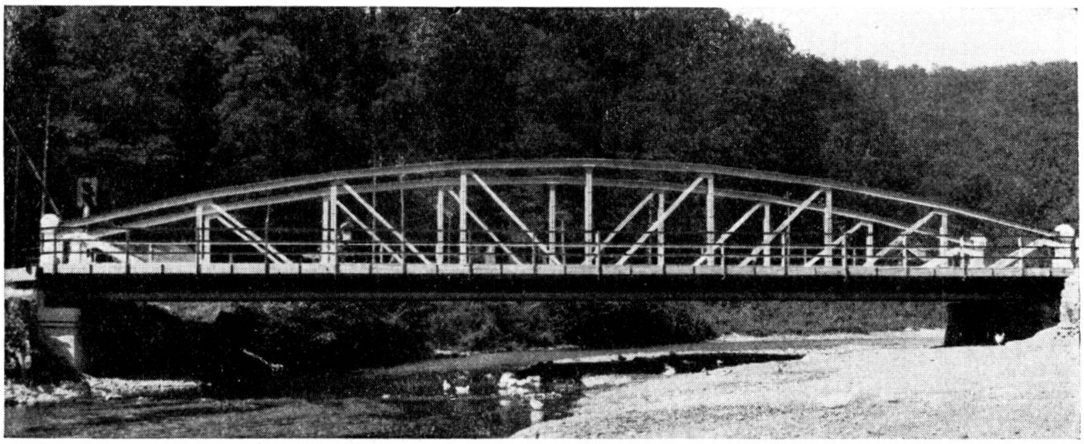

Fig. 6.

In *Switzerland* mention should be made of the road bridge over the Tessin (Fig. 11), a completely welded structure in which the main girders are arches with solid webbed stiffening girders of 70 m span.

Among the hundreds of welded railway and road bridges in *Germany* the following may be named.

1) The bridge over the Ziegelgraben (Fig. 12), forming part of the Rügendam crossing between Stralsund and Dänholm. It is a single track railway bridge

having two fixed spans of 52 m each and one lifting span of 29 m. The material used was St. 37.

2) The single track railway bridge over the Strelasund (Fig. 13), likewise forming part of the crossing of the Rügendam between Dänholm and the island of Rügen, comprises ten openings which are crossed by five continuous plate webbed girders of 54 m span. Here also the material is St. 37.

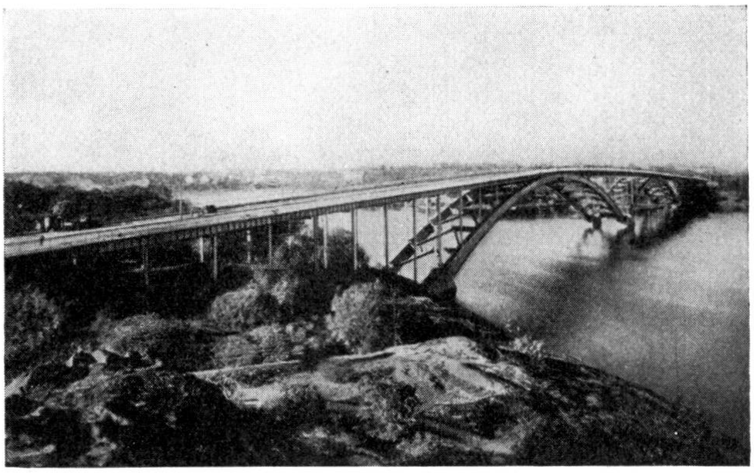

Fig. 7.

3) The bridge for the Reichsautobahn at Kaiserberg near Duisburg (Fig. 14): here the main girders are stiffened arches with plate webbed stiffening girders of 103 m span, and the material is St. 52.

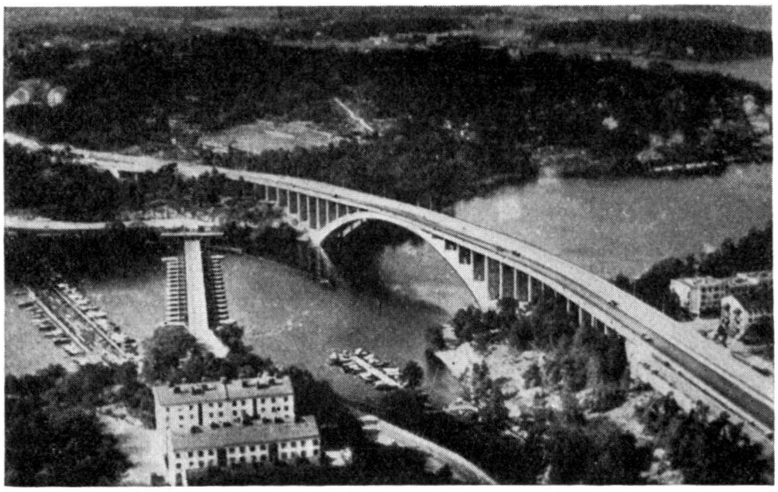

Fig. 8.

4) The Reichsautobahn bridge at Kalkberge (Fig. 15) in the neighbourhood of Berlin. The main girders here are plate-webbed continuous girders of 53.3, 66.7, 66.7 and 63.3 m span respectively, and their depth is 3.0 m, the material being St. 52.

Experience with welded road bridges has been generally favourable, and careful examination of the many welded railway bridges now under traffic in Germany has indicated that their performance in service is excellent, no defects being recorded except insignificant cracks in the web stiffeners.

The main principles to be observed in the welding of bridges are the following:

1) Only those kinds of steel should be welded which are not sensitive to the effects of welding — namely St. 37 and St. 52 which are peculiar as regards chemical composition.

Fig. 9.

2) The mechanical properties of the electrodes must be suitably adapted to the material to be welded.

3) The seams must be limited in number and thickness to the minimum necessary, so as to minimise the production of heat and, therefore, the magnitude of the shrinkage stresses.

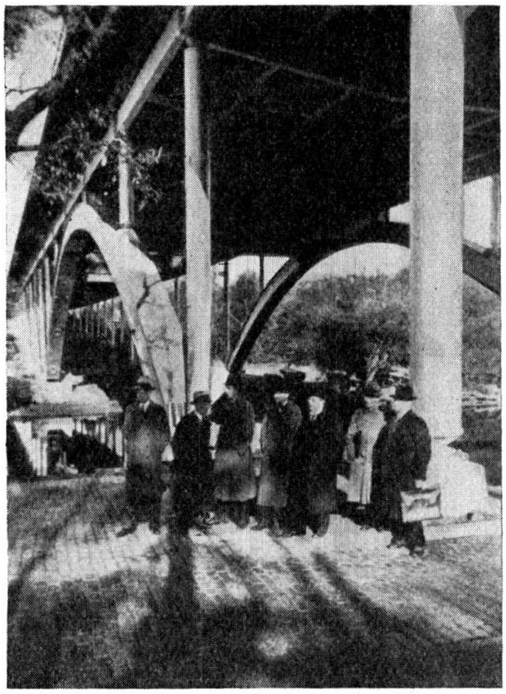

Fig. 10.

4) Web plate joints should take the form of plain butt welds.

5) The necessity for flange joints should be avoided as far as possible by the adoption of long flange plates.

6) Where flange plates have to be connected, the joint should take the form of a plain butt weld or should be made with the aid of cover straps.

7) Butt welds should be placed where the stresses are light (as at points of inflection).

8) The roots of butt welds should be scraped out and carefully re-welded.

9) Butt welds must be X-rayed, and in the case of thick seams this should be done immediately after the first runs of

weld metal have been deposited as experience shows it is in these first layers that cracks occur. For this purpose all fabricating shops which carry out welding work need to be equipped with X-ray apparatus.

10) Seams connecting the web to the flange should also, if possible, be X-rayed by taking samples.

11) Joints should never be made by a combination of welding and riveting. It would be wrong, for instance, to weld the web plate joints and rivet the flange joints, because the slip of the flange joint rivets would have the effect of overloading the web joints.

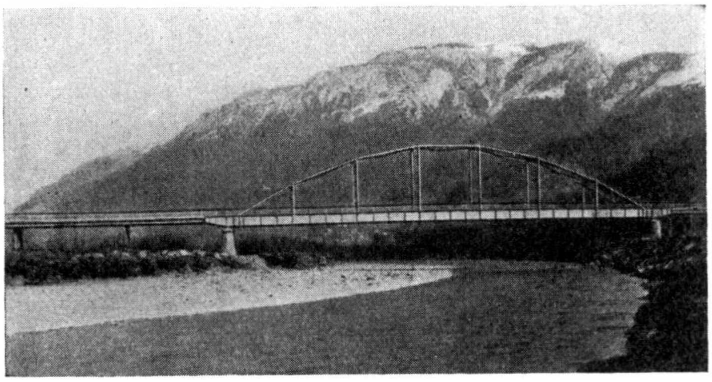

Fig. 11.

12) In tension members the use of transverse seams at heavily stressed points should be avoided. In lightly stressed parts they may be allowed, but must be so arranged as to ensure uniform transition to the parent material.

13) In welding the roadway girders into place care must be taken that no heavy shrinkage stresses arise at the connections. With this object the procedure may be followed, for instance, of first welding the middle cross girder to the

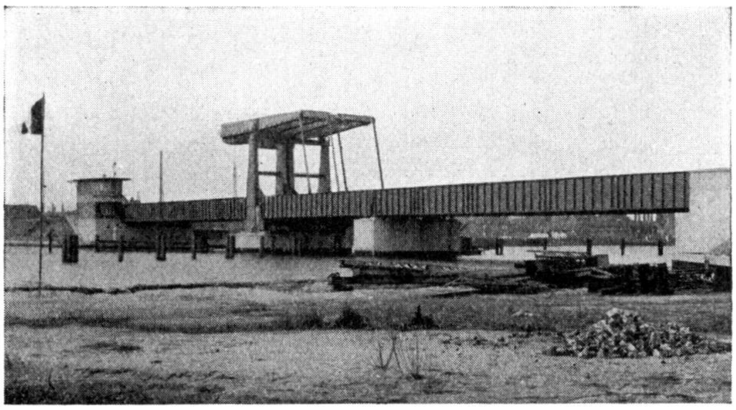

Fig. 12.

main girders, then the adjacent longitudinal girders, next the cross girders on either side of these — hitherto unattached — and finally the last mentioned cross girders to the main girders; and so on.

14) When seams are deposited in several runs, each layer of weld metal must be freed from slag before the next is welded.

15) All welding work, both in the shop and on the site, must be continuously and carefully supervised by competent welding engineers.

16) Welding jobs must be carried out only by tested and reliable welders.

17) The best and simplest welding being done in a horizontal position it is desirable that all important members should be held in suitable rotating supports and so brought into a horizontal position while being welded. Fig. 6 of the following contribution to the discussion IIIb *Dörnen* shows such an arrangement on the site, with a girder in position for welding.

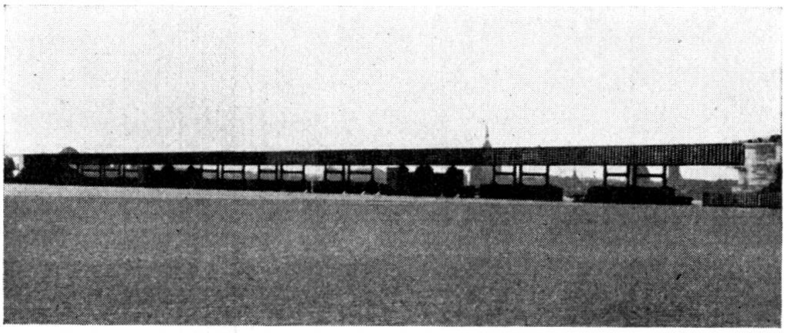

Fig. 13.

Following upon these rules, two kinds of site joint which have been adopted in the construction of plate web girders will be explained:

1) Site joints in the Reichsautobahn bridge at Kalkberge near Berlin already illustrated (Fig. 16). The material is St. 52. The web plate is 3 m high and 20 mm thick, and the flange plates are *Dörnen* reinforced sections 660 $\times$ 44. The joints in the web plate and the flanges are placed at approximately the same positions, corresponding to the points of inflection, and are plain butt joints

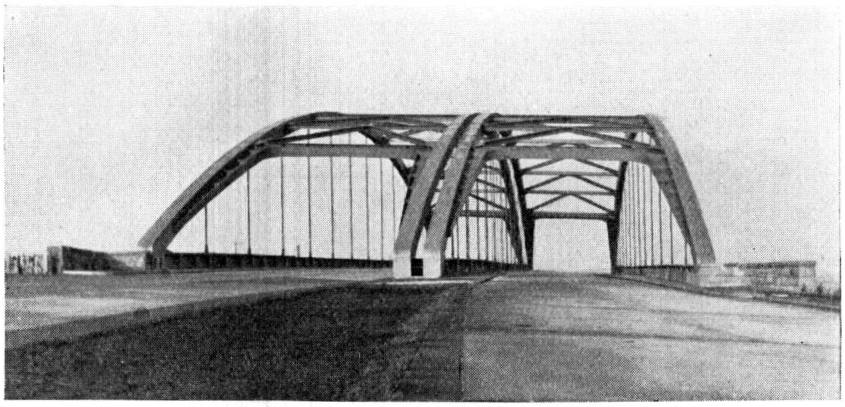

Fig. 14.

taking the form of "tulip" seams in the flanges and of an X-seam in the web plates. The flange joints are placed at 45^0. Five runs were welded in both of the flange joints at the same time, followed by intermittent welding of one side of the X-seam in the web plate from the bottom upwards, simultaneously with the continuation of both seams of the flanges. When one side of the X-seam in the web plate had been completed, the other side was welded. The sequence of welding was so timed that all three seams were completed together.

2) Site joints in the railway bridge over the Strelasund (Fig. 17) which has already been illustrated. Here thicker portions of web plates 40 mm thick are

Fig. 15.

incorporated at the joints with a view to reducing the stresses in the flange plates (*Dörnen* reinforced sections 540 × 55). The web joints thus formed are plain butt welded X-seams and the flange joints were formed immediately between them in the following way:

The flange plates are connected by X-seams at 45°, in addition to which the outside of the flange plates is furnished with small vertical cover straps 150 × 40 mm and horizontal straps 200 × 40 mm above the upper flange and below the lower flange, welded on. The sequence of welding was as follows:

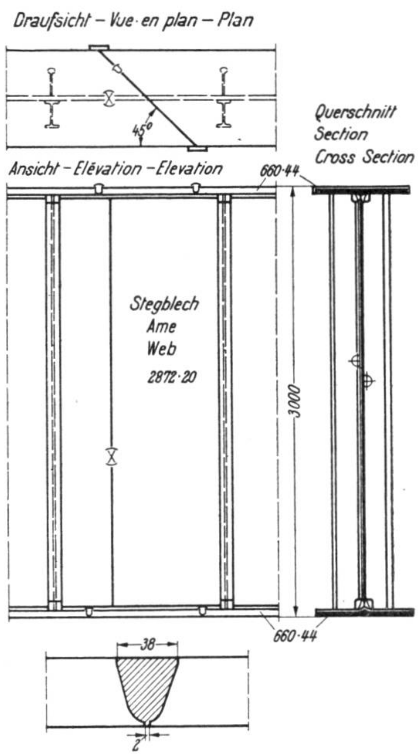

Fig. 16.

The upper halves of the X-seams in the top and bottom flanges were welded at the same time, up to the middle of the height, and were followed by the lower halves of the same seams, welded overhead in the same way. The thickened portion of the web had already been connected to the thinner web plate on one side of it by an X-seam in the workshop, and the remaining butt weld in the web plate was completed after the flange joint already mentioned. Then the butt welds in the flanges were completed, and the neck welds connecting the flange to the web which had previously been omitted opposite the joints were closed. Next the vertical edge covers were attached, and finally the horizontal joint cover plates were welded on.

It is frequently asked why welding is used at all in building work and in bridge construction: why is the well tried method of riveting being given up? Is the idea simply to replace the old by the new?

Undoubtedly every progressive engineer who is not bound by the past feels the urge to create something new, and certainly this is one of the reasons why welding has been introduced into building and bridge construction. The more positive reasons are, however, the following:

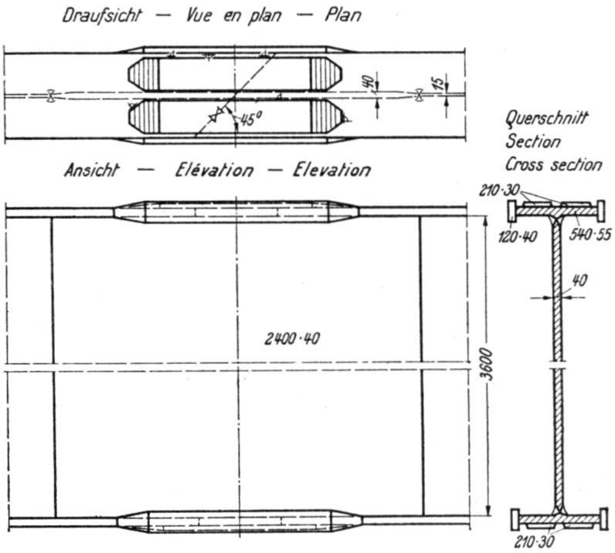

Fig. 17.

1) In general welded girders are more economical than riveted.
2) The appearance of welded structures is better than that of riveted.
3) In the case of plate web girders subject to heavy dynamic stresses a third reason is to be found in the results of the recent tests which show that welded girders possess greater fatigue resistance than riveted.

So far as the economy of welded girders is concerned, it should be observed that in the experience of well equipped steelwork fabricating shops welding is not, in itself, more expensive than riveting. Welded girders are, moreover, usually

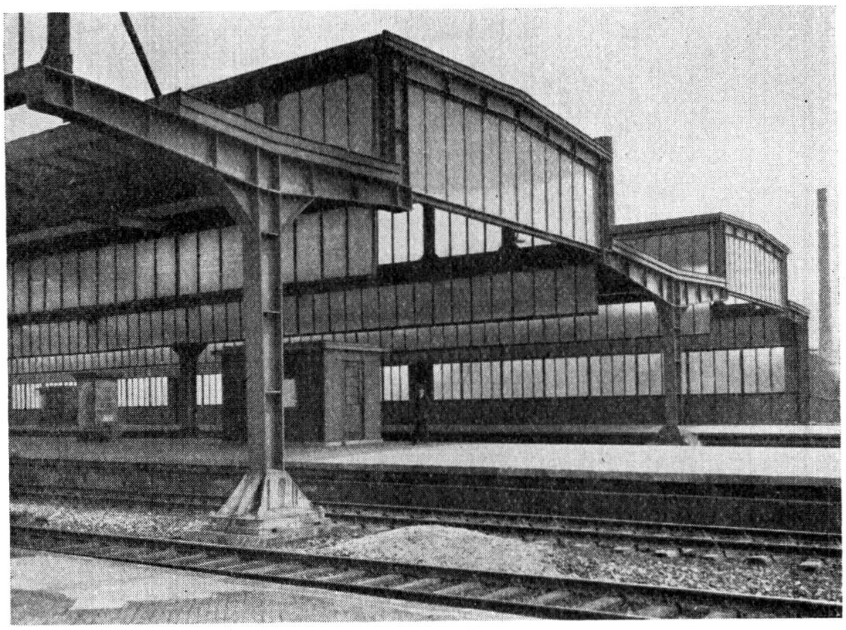

Fig. 18.

16 % lighter than riveted, and there are further incidental savings of even greater importance.

On page 1343 in the "Preliminary Publication" is shown, a two-hinged frame of riveted and of welded construction alternatively, the span and purpose of either design being identical. The riveted frame weighs 19.4 tonnes and the welded frame 14.2 tonnes, so that here there is a saving in weight of 26.3 %.

The following may serve as examples of the extraordinarily good appearance obtained. Fig. 18 shows a welded roof girder for the new railway station at

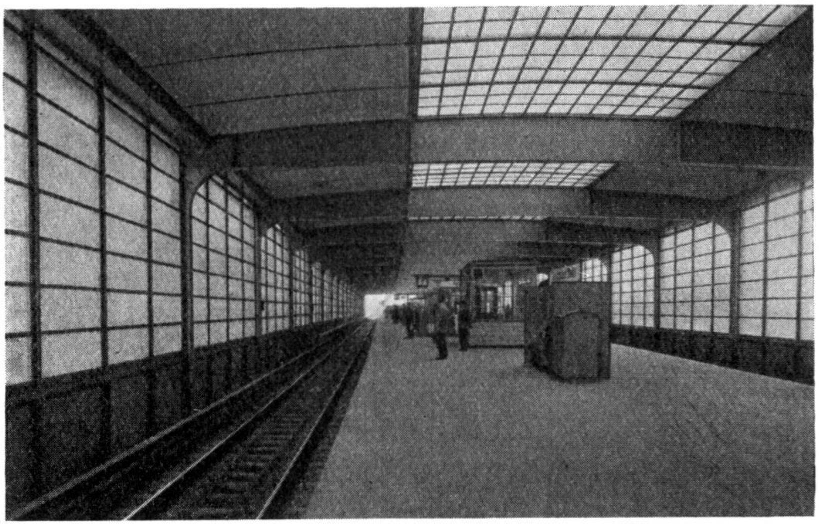

Fig. 19.

Düsseldorf; the pleasing, simple shapes of the kind that appear in this girder and in other portions of the same building cannot be obtained by riveting. Fig. 19 shows the new all-welded tramway station attached to the Zoological Garden railway station in Berlin, and the exceptionally fine appearance presented by

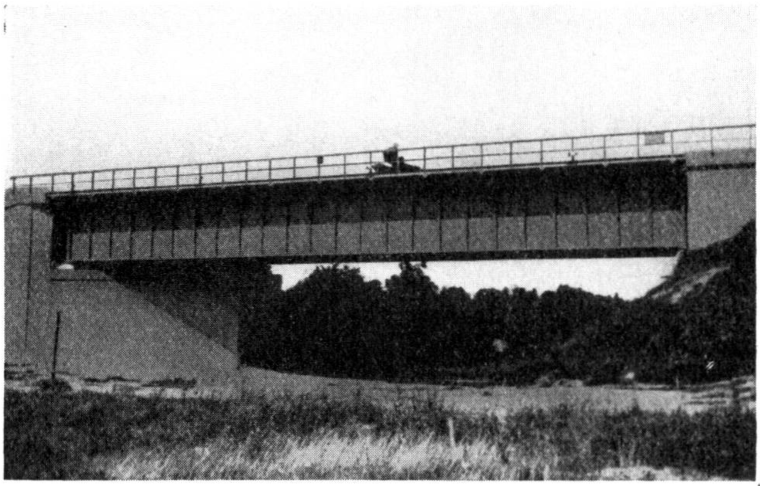

Fig. 20.

welded bridge girders may be appreciated in the railway bridge represented in Fig. 20.

The reasons put forward for the adoption of welding are indeed so convincing that a progressive engineer has every inducement to promote its use in building and bridge work, on the following grounds:

Welding in these applications rests on a scientific foundation underlaid by extensive and careful experiments and research. Good performance has been recorded from welded structures in service. It has been learned how to weld correctly in the proper sequence. It has been ascertained which electrodes and which materials are suitable for welding. Regulations are available for welding in building and bridge work. In the present author's opinion all objections to welding in such work have been overcome and we possess in the X-ray test a simple and dependable means of confirming the qualities of welded seams.

Admitting that many points of detail which need to be clarified by research, are still outstanding it can now be asserted that the use of welding deserves to be promoted as a means of creating structures which are safe in operation, economical and beautiful.

III a

Influence of dynamic and frequently alternating loading on welded structures
(Research work and its practical application).

Einfluß dynamischer und häufig wechselnder Lastwirkungen auf geschweißte Konstruktionen
(Versuchsforschungen und Auswirkung auf die praktische Ausführung).

Influence des actions dynamiques sur les constructions soudées
(études expérimentales et résultats pratiques).

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General Considerations on Welding.

Allgemeine Betrachtungen über das Schweißen.

Considérations générales sur le soudage.

Oberbaurat Dr. Ing. K. Schaechterle,
Direktor bei der Reichsbahn, Berlin.

The results obtained between 1928 and 1933 by Professor *O. Graf* in his experiments on fatigue in the Materials Testing Laboratory of the Technical University of Stuttgart have been summarised in a report by the present writer entitled "On fatigue strength of rivetted and welded joints and the design of dynamically stressed structural members based on conclusions drawn from fatigue tests" (I.A.B.S.E. Zurich, 1934). Since that time our knowledge of the effects of dynamic and frequently repeated loading on rivetted and welded members has been extended and deepened by a great number of researches.

It has been found, in agreement with parallel investigations in other research laboratoires, that the fatigue resistance of structural steels of different compositions and origins is affected by irregularities in the material, by surface conditions and by the treatment received in the workshop. It has likewise been found that unfavourable effects in this respect may be associated with the presence of contractions or enlargements of the cross section, as well as holes and weld seams, and that when fracture occurs under steadily increasing static loads, it is preceded by a considerable amount of plastic deformation, which in turn is connected with the phenomenon of cold working. Under a condition of pulsating or alternating stress, many times repeated, the plastic deformation and the hardening brought about by the cold working effect are much reduced, and the fatigue stress value may lie a good deal lower than the normal breaking stress value. Under static loading, local increases and concentrations of stress may be mitigated by the equalising action which occurs in the plastic region but under dynamic loading such local concentrations of stress tend to manifest themselves in a reduction of the fatigue resistance. This effect is more marked in hard than in soft steels, and those in which an increase in the breaking strength or yield point is accompanied by a large increase in the susceptibility to notching are unsuitable for use in bridge construction.

Test bars of St. 37 and St. 52 respectively gave tensile strengths of $\sigma_B = 40$ and 57 kg/mm², and surge load limits of stress $\sigma_U = 27$ and 31 kg/mm². The ratio of the surge load limit of stress to tensile breaking stress is 0.68 for St. 37 and 0.55 for St. 52, the corresponding value as obtained on bars with holes in them being 0.50 for St. 37 and 0.36 for St. 52. The reduction is greater in the case of St. 52 than in that of St. 37, a fact which may be attributed to the former being more susceptible to notching.

The usual grade of structural steel, St. 37, is equally adapted for rivetting and welding, but high tensile steels of St. 52 grade show considerable differences as regards weldability according to their compositions and origins. High carbon or alloy steels which have been incorrectly treated either mechanically or thermally have been known to result in failures. Thus a steel St. 52 containing 0.25 % C, 1.20 % Mn, 0.76 % Si, 0.5 % Cu, 0.023 % S and P, which gave the high tensile strength value of $\sigma_B = 58\ kg/mm^2$ and a yield point $\sigma_F = 44\ kg/mm^2$ with 20 % elongation, nevertheless showed very fine cracklike notches radiating from the edge and these developed into dangerous cracks when the usual unavoidable shrinkage stresses occurred on the steel being welded. In steels of grade St. 52 having smooth surfaces free from notches and containing less than 0.18 % C and less than 0.5 % Si these effects have not hitherto been observed.

It follows, therefore, that if St. 52 is used for welded structures its optimum composition must be confirmed by experiment. Another matter calling for consideration is the choice of the correct welding rod to give the best welded joint on the parent material in question, and this is best decided by using the commonest forms of electrodes to make butt welds in test bars wich can be subjected to fatigue tests to determine the surge load limit of stress. The composition and crystal structure, both of the parent metal and the weld metal, should be ascertained, and also their liability to cracking when welded.

I. The first German Regulations for Welded Steel Structures (DIN 4000 for bridges and structural work) appeared in 1931. These were based on statical tests carried out in the material testing laboratory at Dresden. At that time a connection made with side fillet welds was considered more reliable than a butt joint, and the joints were, therefore, covered with fish plates after the manner of rivetted work; in other respects also welded bridges were treated in the same way as the accepted forms of rivetted bridges, the same formulae being used in each case to determine the cross sections. For instance under alternating stress with St. 37 the stress was calculated as $\sigma = \frac{M_{max} - 0{,}3\, M_{min}}{W_n} \leqq \sigma_{zul} \leqq 14\ kg/mm^2$. For weld seams in the region of pulsating stress the American formula $M = \max M + \frac{1}{2}(\max M - \min M)$ was introduced.

The earliest fatigue tests to be carried out on welded structural elements showed surprising results by comparison with the static tests. It was found that the efficiency of a welded connection depended not only on the mechanical properties of the weld, which were influenced by the thermal effects of the welding process on the properties of the parent material and transition zone, but also, to a still greater extent, on the flow of stress, and that the principal criterion governing breakage through fatigue was in fact the internal distribution of stress.

Hence the butt joint which allows an undisturbed flow of stress shows considerably better values of "pulsating" strength than the strapped joint with side fillets which involves deviations in the flow and concentrations of stress. Consequently, if butt connections were reinforced by straps with fillet seams the fatigue strength was found to be considerably reduced, whether against alternating or pulsating loads, and the originating crack for fatigue breakage was found to start from the end fillets. Moreover, it was found that light concave fillet welds, causing a gradual transfer of stress from the weld to the plate, gave better results than full fillet welds, contrary to what had hitherto been thought.

On the basis of these experimental results it was prescribed in DIN 4100, issued in 1931, that a butt weld might be used with four-fifths of the permissible stress of the parent metal. This decision led to the development of oblique seams, but it was found in later experiments that the lengthening of the weld seam in this way did not enable the fatigue strength of the connection to be increased by comparison with a square butt seam. On the other hand, where cover straps to the connections were provided it was found possible to increase the fatigue strength by taking suitable constructional precautions such as by bevelling the ends of the straps in order to ensure that the change in the cross section should be gradual, and by careful after-treatment of the fillet seams by milling, particularly at the beginning of the side fillets. In accordance with these facts the regulations had to be again amended and redrafted, and the occasion was seized to introduce a fundamental distinction between structures carrying static stresses and those carrying mainly dynamic stresses.

If the fatigue strength of either an ordinary or a high tensile structural steel is plotted in a diagram the result is the σ_o line shown in Fig. 1, referred to lower stress value σ_u. The amplitude $\sigma_S = \sigma_O - \sigma_U$ is determined from the distance between σ_O line from a line inclined at 45°. The shaded area represents the "pulsating" zone, those portions of the diagram which extend beyond this limit being ruled out for practical purposes, because the deformations imply that the yield point or crushing stress is exceeded. In the remaining region of the diagram the curve of fatigue strength may be approximately represented by a straight line. The amplitude decreases somewhat in the region nearing the yield point. According to the experiments of Professor *Graf* it is amply accurate enough to assume that the amplitude σ_S is of uniform amount throughout the tensile and compression zone for welded constructions; in other words it may be assumed that $\sigma_S = \sigma_U = 2\,\sigma_W$.

The factors governing the dimensions of bridge members subject to traffic loads, which cause pulsating stresses, are the respective maximum values of the normal forces, the shear forces and the bending moments under the influence of the stationary load and of the live load multiplied by the impact coefficient φ. The stresses calculated from these maximum values must fall within the safe limits as indicated in the region $\sigma_{D\,zul}$.

On the basis of experiments recorded in *Dauerfestigkeitsversuche mit Schweiß-verbindungen* (Berlin, 1925, VDI-Verlag) the limits of stress as adopted by the Drafting Committee for Regulations for Welded Railway Bridges (on which both scientific research and practical interests are represented) are those indicated in Figs. 2a and 2b.

There is an unmistakeable tendency to force the permissible stresses to a maximum, and the values of $\sigma_{D\,zul}$ are in fact very close to the alternating stress values as determined by experiment (average values for 2000000 alternations of stress), with the result that the available margin of stress is at places only

1 kg/mm² even when no account is taken of the unavoidable shrinkage stresses. Retaining the methods of calculation prescribed by the "BE" (Rules for calculating steel bridges) a variable "shape coefficient" α is introduced in addition to the vibration factor γ appertaining to an unjointed member, in order to allow

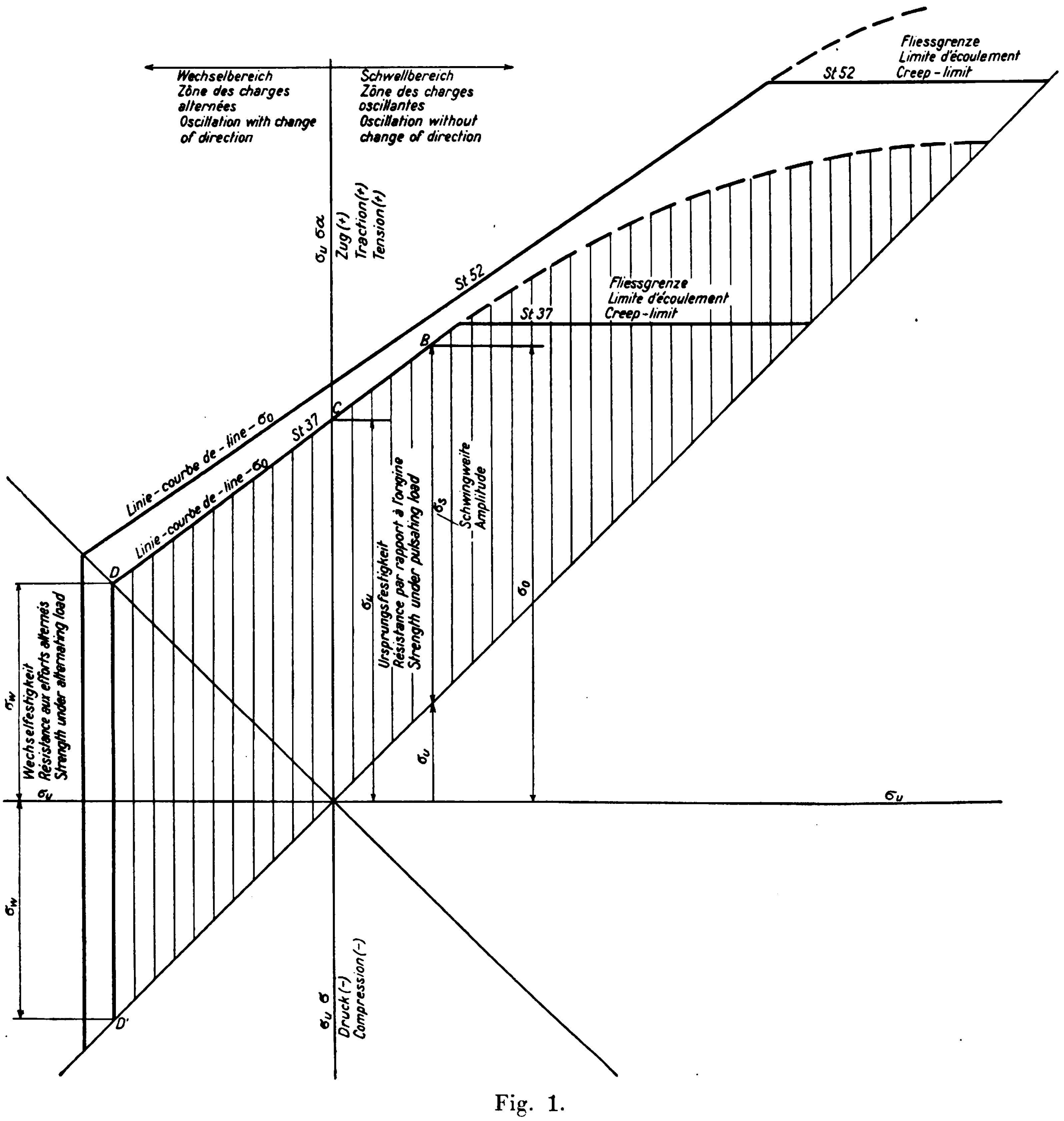

Fig. 1.

Curve of fatigue strengths σ_o for St. 37 and St. 52 in tension.

of treating a dynamically loaded member in the same way as a member subjected to stationary loading. According to $\sigma_1 = \frac{\gamma}{\alpha} \cdot \frac{\max M_I}{W_n} \leq \sigma_{zul}$ the maximum values of bending moments ($M_I = M_g + \varphi \cdot M_p$) for dead load and live load respectively are to be multiplied by the coefficient γ and again multiplied by the coefficient α according to the nature of the construction and that of welding. Thus the calculation is performed on the basis of imaginary stresses σ_I, whereas the stresses which actually arise are given by $\sigma_{max} = \frac{\alpha}{\gamma} \cdot \sigma_I \left(= \frac{\max M_I}{W_n}\right)$.

The values of γ and α applicable to railway bridges are shown in Tables 1, 2 and 3 of the "Provisional Regulations for Welded Solid-Webbed Railway

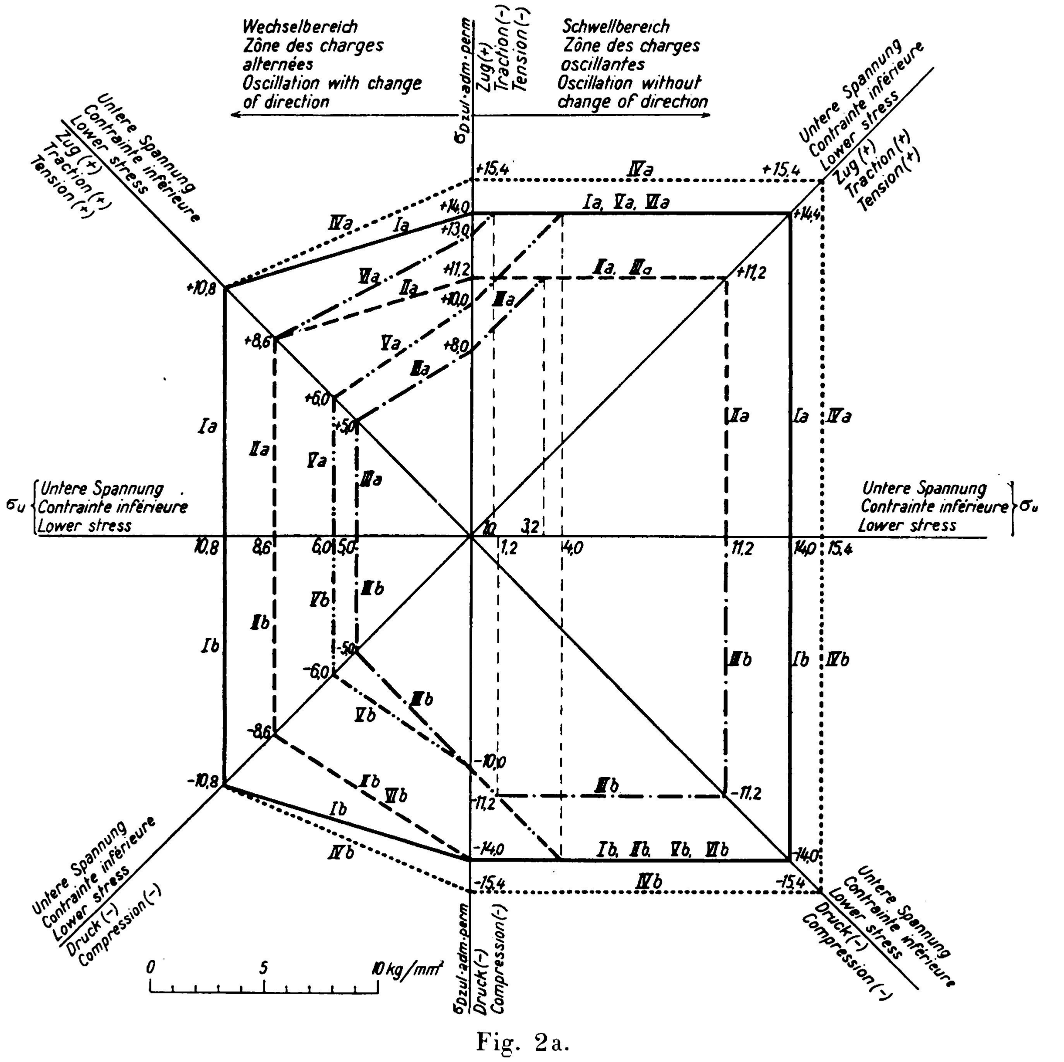

Fig. 2a.

Diagram of permissible stresses σ_{Dzul} in welded bridges of St. 37:

Ia, Ib	— unjointed members in tensile and compressive zones.
IIa	— jointed members in tensile zone, close to butt welds and in the butt welds themselves, when the roots of the seams have been re-welded and the seams worked over.
IIb	— same as IIa, in compressive zone.
IIIa, IIIb	— same as IIa and IIb when it is not possible to re-weld at the root.
IVa, IVb	— permissible principal stresses according to the formula

$$\sigma = \frac{\sigma_I}{2} + \frac{1}{2}\sqrt{\sigma_I^2 + 4\tau_I^2}.$$

Va, Vb	— members close to end fillet welds and at the beginnings of side fillet welds, the transitions to the former and the ends of the latter not being worked over.
VIa, VIb	— same as Va and Vb with optimum treatment of the transitions to end fillets and ends of side fillets.

Bridges" (*Vorläufigen Vorschriften für geschweißte vollwandige Eisenbahnbrücken*). The values of γ, which depend on the ratio $\frac{\min S_I}{\max S_I}$ or $\frac{\min M_I}{\max M_I}$ may be

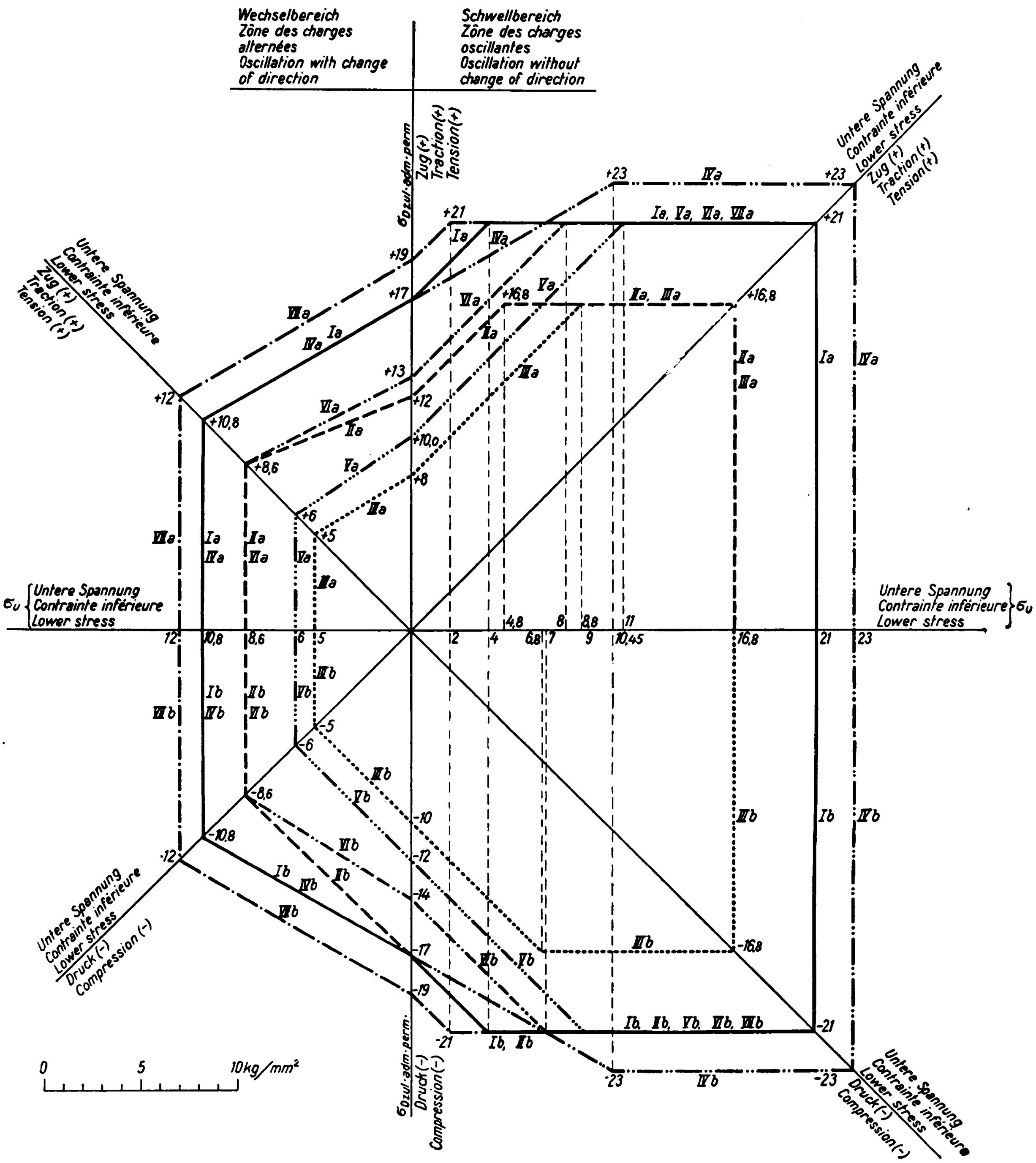

Fig. 2b.

Diagram of permissible stresses σ_{Dzul} in welded bridges of St. 52.

Ia, Ib — unjointed members in tensile and compressive zones under heavy traffic (more than 25 trains a day on each track).

IIa — jointed members in tensile zone, close to butt welds and in the butt welds themselves, when the roots of the seams have been re-welded and the seams worked over.

IIb — same as IIa, in compressive zone.

IIIa, IIIb — same as IIa and IIb when it is not possible to re-weld at the root.
IVa, IVb — permissible principal stresses according to the formula

$$\sigma = \frac{\sigma_I}{2} + \frac{1}{2}\sqrt{\sigma_I^2 + 4\tau_I^2}.$$

Va, Vb — members close to end fillet welds and at the beginning of side fillet welds, the transitions to the former and the ends of the latter not being worked over.
VIa, VIb — same as Va and Vb with optimum treatment of the transitions to end fillets and ends of side fillets.
VIIa, VIIb — same as Ia, Ib under light traffic (up to 25 trains a day on each track).

inferred from the stress diagram for $\alpha = 1$. The values of α in relation to $\frac{\min S_I}{\max S_I}$ or $\frac{\min M_I}{\max M_I}$ may appear as slightly curved lines, which in the regulations are represented approximately by straight lines.

This form of calculation does not lead to simplification of the design. The reduction in the permissible stresses, clear and un ambiguous in the stress diagrams, is somewhat obscured by the introduction of the coefficients. The natural procedure is to check that the maximum stress σ_{max} as obtained from the statical calculations always falls within the portion of the diagram indicated as permissible. Through the reduction imposed on the value of the permissible stresses, the designing engineer is guided to adopt economical and correct design from the point of view of welding: for instance to adopt butt welds instead of cover straps, to place the joints of girders in the neighbourhood of the points of inflexion of the moments, and to separate the top and bottom welds of plate web girders from the flange plates.

If the amplitude is assumed to have the same values throughout, the general formula for the amplitude falling off in accordance with the yield point or crushing stress, takes the following form:

$$\sigma_{D\,zul} = \frac{\sigma_{U\,zul}}{1 - \frac{\sigma_{U\,zul} - \sigma_{W\,zul}}{\sigma_{W\,zul}} \cdot \frac{\min S}{\max S}} = \frac{\max S}{F_{erf}}$$

With $\sigma_S = \sigma_U = 2\,\sigma_W$ we obtain the simple design formula

$$F_{erf} = \frac{\max S - \min S}{\sigma_{U\,zul}}$$

as given originally by *Wöhler*. In this way the process of calculation is greatly simplified and there is no necessity to make use of tables. It is only necessary to know the basic stresses, that is to say the permissible amplitudes $\sigma_{S\,zul}$ and the reduction coefficients for a butt joint (= 0.8) and for fish plated connections with end and side fillet seams (0.65 — 0.75) respectively. If these values are borne in mind design may be carried out without reference to tables.

Under these simplified assumptions (according to present knowledge) the lines Ia and Ib are applicable to an amplitude of $\sigma_{S\,zul} = \sigma_{U\,zul} = 2\,\sigma_{W\,zul} = 14$ or 16 kg/mm^2 respectively for tension or compression in continuous unjointed members made of St. 37 or St. 52. Similarly the line IIa applies to butt jointed members (wherein the root of the seam has been welded over and wherein transitions of the seam have been after-treated) for the tensile zone with an amplitude of $0.8 \times 14 = 11.20$ kg/mm² for St. 37 or $0.8 \times 16 = 12.80$ kg/mm² for St. 52. Butt welds in the compression zone may be stressed to the same extent as continuous members without joints (line IIb). In the case of members in the proximity of end fillet seams and at the beginning of the end of side fillet seams (with the transitions carefully worked) the lines IIIa and IIIb are applicable ($0.75 \times 14 = 10.5$ kg/mm² for St. 37, $0.75 \times 16 = 12.0$ kg/mm² for St. 52). If, however, the seams are not worked, the lines to use are IVa and IVb ($0.65 \times 14 = 9.1$ kg/mm² for St. 37; $0.65 \times 16 = 10.4$ kg/mm² for St. 52). (Figs. 3a and 3b.)

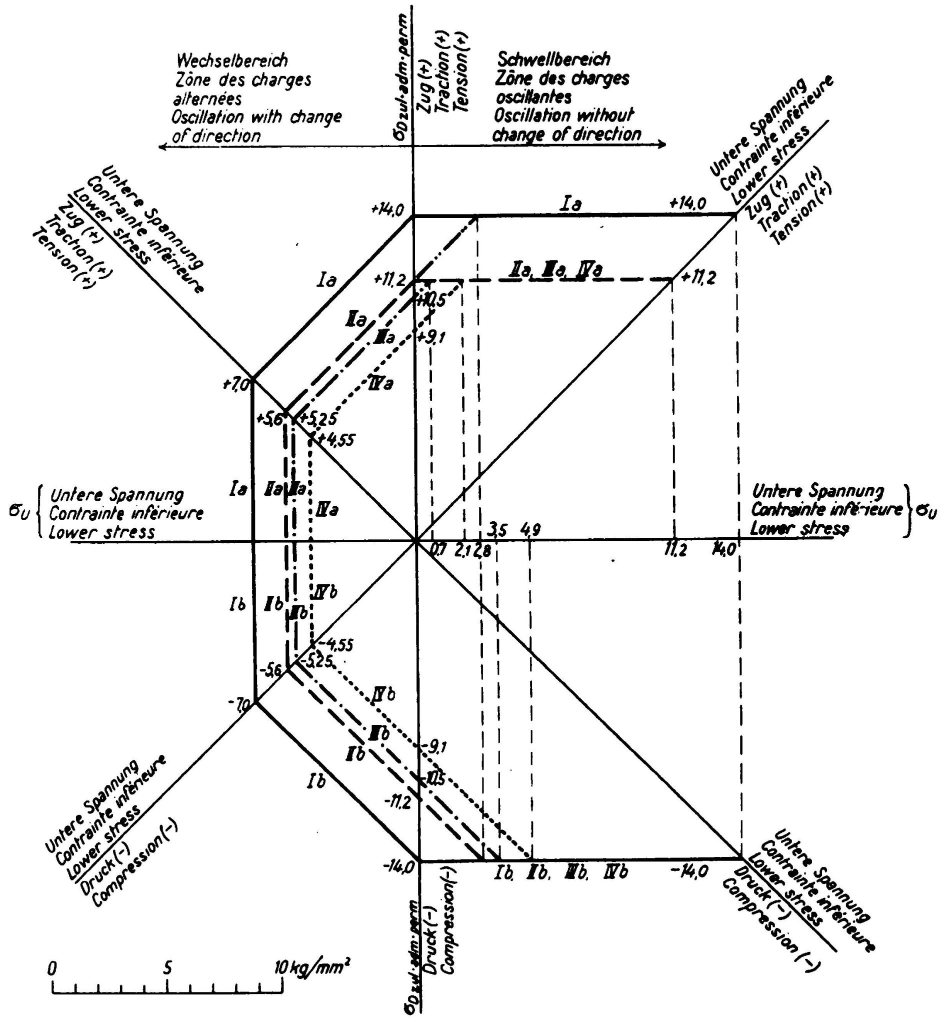

Fig. 3a.

Diagram of permissible stresses $\sigma_{D_{zul}}$ in welded bridges of St. 37.

Ia, Ib — unjointed members in tensile and compressive zones $\sigma_u = 14$ kg/mm².

IIa, IIb — jointed members in tensile and conpressive zones, close to butt welds and in the butt welds themselves, when the roots of the seams have been re-welded and the seams worked over. $\sigma_u = 0{,}8 \times 14 = 11{,}2$ kg/cm².

IIIa, IIIb — members in tensile and compressive zones close to end fillet welds and at the beginnings of side fillet welds, the transitions to the former and the ends of the latter being carefully worked over. $\sigma_u = 0{,}75 \times 14 = 10{,}5$ kg/mm².

IVa, IVb — same as above, in tensile and compressive zones, transitions to end fillets and ends of side fillets not being worked over. $\sigma_u = 0{,}65 \times 14 = 9{,}1$ kg/mm².

II. Turning now to the application of this research work to practical design, the fatigue tests have made it possible to make better use of the properties of the material by careful design and in this way to save weight, while at the same time

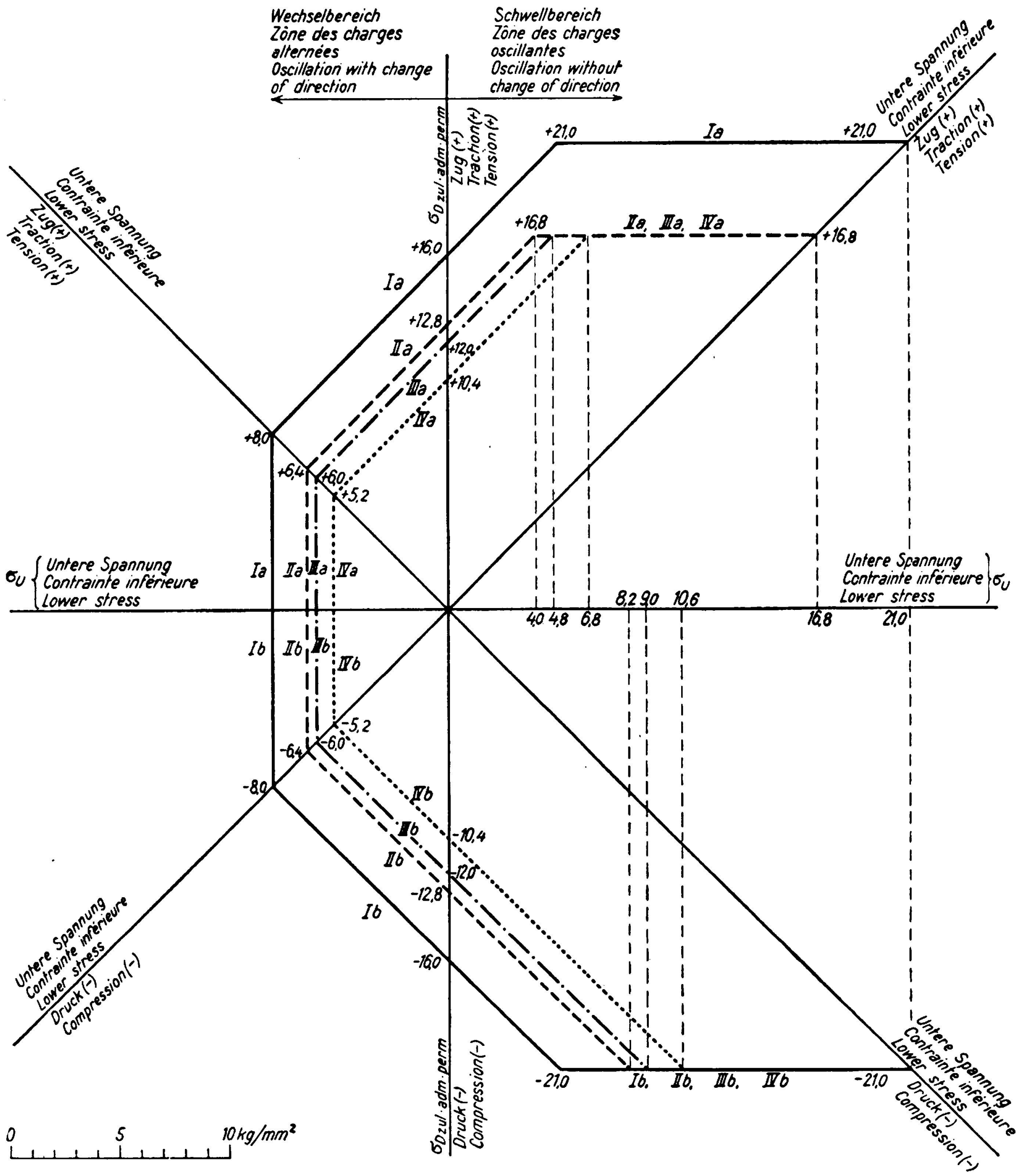

Fig. 3b.

Diagram of permissible stresses $\sigma_{D\,zul}$ in welded bridges of St. 52.

Ia, Ib — unjointed members in tensile and compressive zones. $\sigma_u = 16$ kg/mm².

IIa, IIb — jointed members in tensile and compressive zones, close to butt welds and in the butt welds themselves, when the roots of the seams have been re-welded and the seams worked over. $\sigma_u = 0{,}8 \times 16 = 12{,}0$ kg/mm².

IIIa, IIIb — members in tensile and compressive zones close to end fillet welds and at the beginnings of side fillet welds, the transitions to the former and the ends of the latter being carefully worked over. $\sigma_u = 0{,}75 \times 16 = 12{,}0$ kg/mm².

IVa, IVb — same as above, in tensile and compressive zones, transitions to end fillets and ends of side fillets not being worked over. $\sigma_u = 0{,}65 \times 16 = 10{,}4$ kg/mm².

increasing the safety and reliability of the welded structure. The design of welded connections on the basis of statical tests has often led to incorrect conclusions and faulty construction, and in this respect it may be remembered that proposals were put forward to strengthen rivetted connections by side fillets and butt welds to cover by welded — on fish plates; but the real improvement in correct welding is especially due to fatigue tests. The first rule to be followed in correct welding design is to avoid everything which may prejudice fatigue resistance.

The knowledge obtained from fatigue experiments has led to a change in the features of welded structures; the designer of welded work has freed himself to an increasing extent from rivetted forms. He has learnt to produce bridges from the smallest possible number of easily weldable elements; to avoid undesirable multiplication of weld seams, deviations of the flow of stress through cover straps and sudden changes of direction, concentrations of stress due to sudden changes in cross section-or at any rate to reduce these features to a minimum.

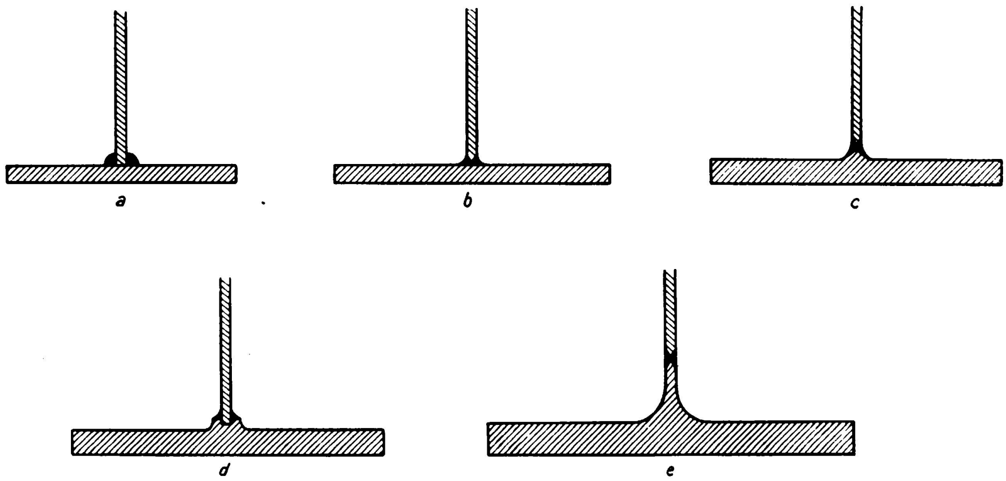

Fig. 4.

And finally, he has learned to make the weld seams as short and small in cross sections as possible with a view to minimising the anuvoidable thermal and shrinkage stresses.

The most important structural form in bridge work at the present time is the solid-webbed girder made by welding together plates and special rolled sections of differing widths and thicknesses. The suitability and economy of the plate web girder has been proved numerous examples, and through the work of *Dr. Schaper* girders of this type have been built up to 54 m in span (for instance in the Rügendamm bridge).

Welded girders of I-section, built after the manner of a rolled joist, are used with advantage in small and medium span of bridges. This type gives a freer choice of dimensions, not being limited by considerations of rolling, as is the joist type of bridge. Besides the I-section, use is made of box and other hollow cross sections.

In the earliest welded plate web girders (Fig. 4) the flange plates were connected to the web plate by full fillet seams (Fig. 4a), but experience and research have led to the bevelling the edges of the web plate and welding with concave fillet seams on either side (Fig. 4b). With thick web plates this, however, led to difficulties in ensuring penetration at the root of the weld, and the further step was taken of adopting special shapes of rolled section for the flange plates, among which may be mentioned the "nosed section" *(Nasenprofil)* of the Dortmunder Union for use with fillet seams (Fig. 4d), the spine-plate *(Wulstprofil)* of *Dr. Dörnen* (Fig. 4c), and also Krupp's ribbed plates which are butt-welded to the web plate. The displacement of the weld connecting the web plate to the flange plate into a region of lower stress, and the improved stress conditions brought about by the more uniform transition, are among the advantages obtained in the last mentioned, while at the same time notching effects in the flanges is avoided (Fig. 4e). There is the further advantage that bringing the seam some distance away from the flange renders it accessible for examination, and it can be easily tested by X-rays.

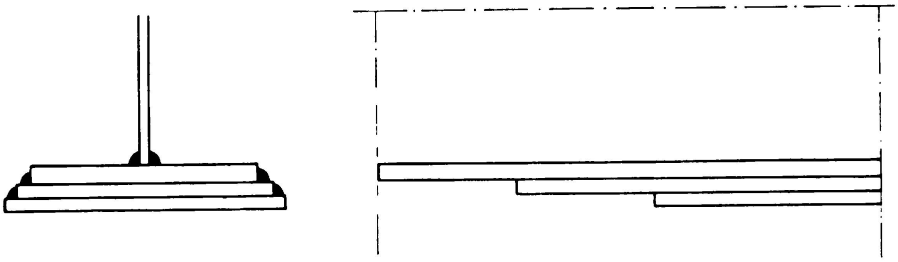

Fig. 5.

For economic reasons it is not possible, in girders of long span, to avoid the necessity for curtailing the cross section of the flanges in accordance with the bending moment diagram. In such cases the additional flange plates have been stacked one above the other, and welded by means of fillet seams (Fig. 5). From a constructional point of view this arrangement is open to some objection, because only the innermost plate of the stack is connected to the web plate and

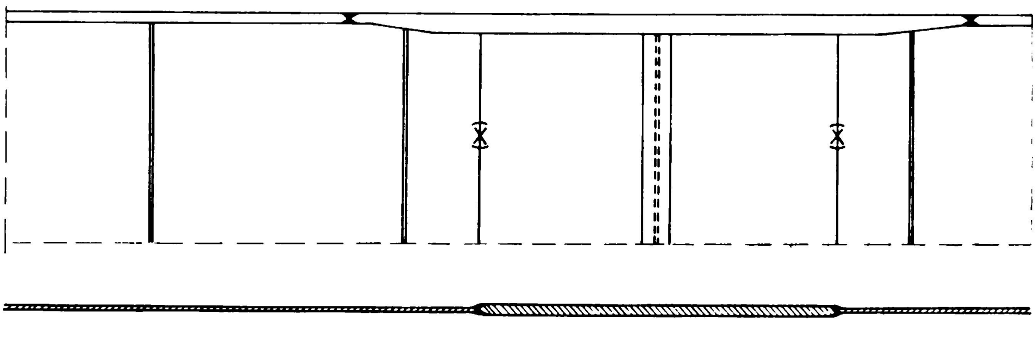

Fig. 6.

fillet seams are inadequate as a means of connecting wide plates; when the fillet seams are welded the wide plates tend to arch, and in this way the risk of buckling in the compression zone is increased. The disadvantage is reduced if the flange plates are made of differing thicknesses and are connected by butt welds so as to give a gradual transition (Fig. 6). Usually the thickening of the flanges is placed on the inside so as to obtain flat surfaces on the outside.

A good arrangement is obtainable by the use of nosed and ribbed sections with reinforcing plates welded onto the inside (Figs. 7a to c). If this design is adopted the rib of the web plate should be made high and strong enough to make the weld easily accessible and to allow of the additional plate for strengthening the flanges being connected to the web plate by butt welds; in

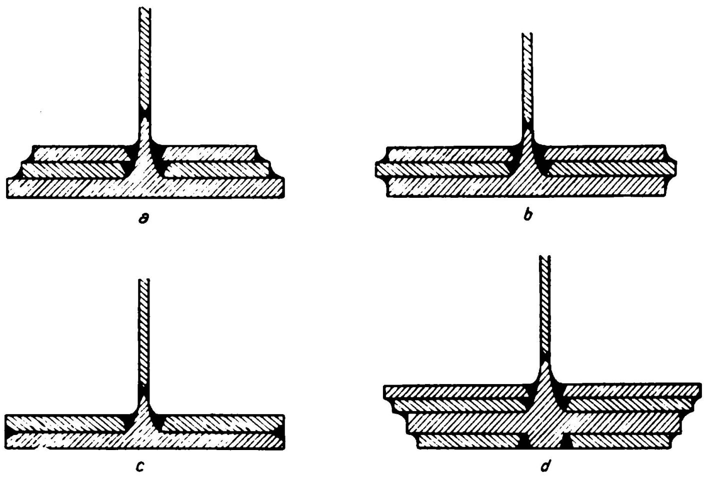

Fig. 7.

this way a perfectly uniform flow of stress from the flanges to the web may be ensured. A further improvement towards making the cross section of the flanges conform to the curve of maximum moment is obtained by the use of cross rib plates (Fig. 7d) with additional flange reinforcing plates connected to the ribs

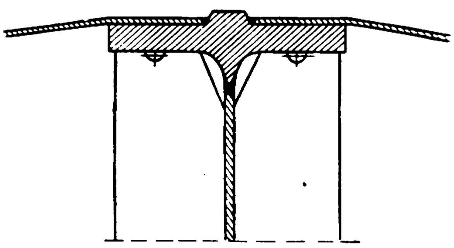

Fig. 8.

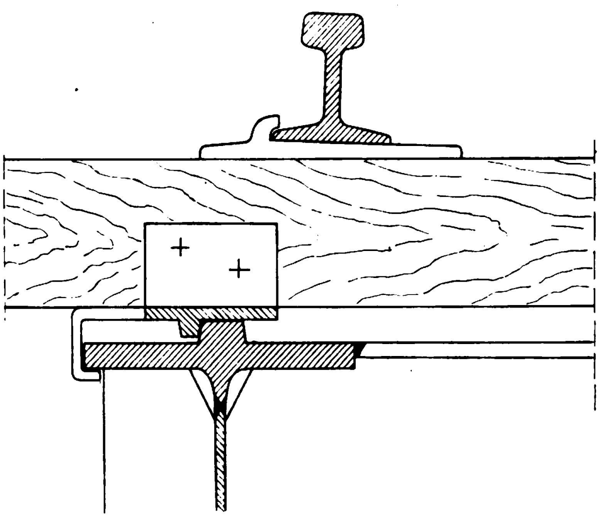

Fig. 9.

by butt welds on either side. In the case of road bridges it is advantageous to make use of trough plates or buckler plates welded into position by butt seams (Fig. 8), and in railway bridges the adoption of cross ribbed plates enables the sleepers to be arranged centrally (Fig. 9).[1]

[1] *Schaechterle:* Der geschweißte Vollwandträger. Beitrag zur Gestaltung von geschweißten Brücken. Bauingenieur, Vol. 17, Nos. 15/16, pages 131 follg.

Another important consideration in the design of welded plate web girders is the stiffening of the sides of the web. Where the latter is of considerable depth the stiffening may be provided by flat plates, T or I sections. If the web plate is more than 16 mm thick it is not necessary that the seams should be staggered, but to avoid crossing the welds which connect the flange to the web the inner corners of the stiffeners are cut away. In the tension zone packing plates are driven in, and are connected to the stiffeners by fillet welds in order to prevent any weakening of the tensile flange by notches burnt out. In deck bridges it is advisable to connect the upper flange of the cross girders or cross frames to the main girders by butt welds, and also to weld the web plates of the cross girders to the main girder completely at the corner of the frame; in this way the forces in the flange of the cross framing will be transferred to the main girder and there will be no lack of rigidity (Fig. 10).

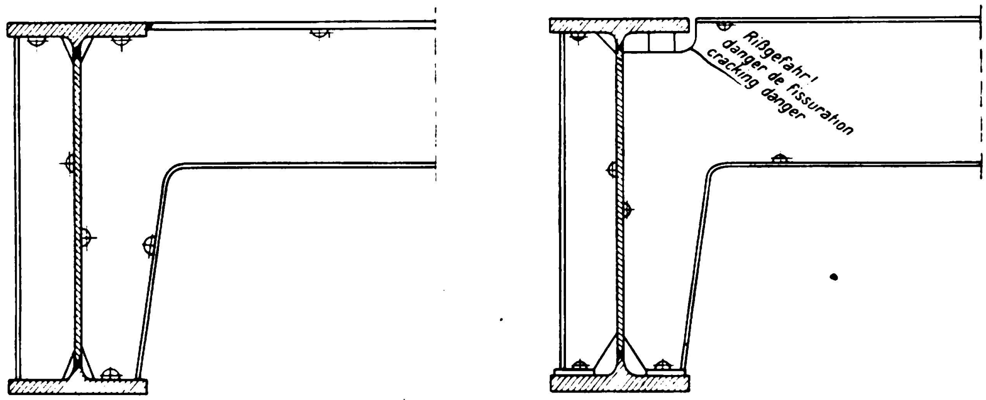

Fig. 10.

III. A problem which still awaits solution is that of the shrinkage stresses which arise in welding. In the regulations no account is taken of these, although it is a matter of experience that they may reach high values, and where the design or sequence of welding are faulty may even lead to cracking. The study of shrinkage stresses has been somewhat neglected even in research, and the next task to be undertaken must be that of ascertaining the magnitude of these shrinkage stresses in bridges, and their effect on the fatigue resistance. The existence of internal stresses in welded structures is a danger, if by the welding operation fine cracks are generated.

Recorded failures are attributable to the following causes:

1) High carbon or high silicon welds with defects, such as very fine notches in the form of cracks, pores, laps, doublings, etc.

2) Faulty workmanship, such as the cold bending of stiff sections.

3) Unsuitable heat treatment, with the result that cooling and shrinking stresses are superimposed on one another.

4) Concentrations of stress due to weld seams being crowded together, sudden changes of cross section, etc.

It is necessary to learn first of all to master shrinkage stresses in actual practize, and from this point of view the welding process itself should be properly planned on the basis of experience and research.

Shrinkage stresses arise when the free movement of structural parts which are being welded together is hindered. Up to a certain point shrinkage stresses can be eliminated by cold stretching, but the conditions are not the same as those in joists subjected to rolling and cooling stresses, and under unfavourable conditions the shrinkage stresses in welds may reach considerably higher values. Hence every precaution must be taken in the welding of bridge work to reduce the shrinkage stresses to a minimum, and these precautions should begin even in the selection of plates or pieces of material for the welded connections, which should be governed by the possibility of forming perfect weld seams. Since plates less than 8 mm thick are only exceptionally used in bridge construction the continuous one layer welding is not practised. For forming V, X or U seams the abutting edges of the plate should be carefully prepared so as to ensure that the cross section of the seam will extend uniformly over the whole length and that thickenings of weld metal will be avoided. The welding gap should be at least 2 mm, in order to ensure perfect penetration of the weld at the root and to avoid too deep scraping out of the root when finishing the seam afterwards. While the seams are being formed, the different parts of the work must be so arranged that movement in the principal direction of shrinkage is freely possible, while at the same time deformations are limited to a minimum. In selecting the welding rods the thickness of the plates to be connected must be considered; the thicker the plates the thicker the electrodes to be used. The risk of cracking is greatest while depositing the first layer of the seam, and great care should, therefore, be exercised when making the first run using an electrode too thin. The butt joint with a V seam is adopted for thin plates, and also for thick plates where it is not possible to turn the work over.

On thick plates the U seam has the advantage over the V seam of requiring a smaller amount of weld metal. Either kind of seam allows of the root being cleaned out and rewelded overhead. In both cases the matter of angular distortion requires attention, and this may be met by the precaution of placing the two plates to be joined at a small angle to one another so that after the angular distortion has taken place they will lie in the same plane. The X form of seam is the one which requires the smallest amount of weld metal and is used where the welding can be performed in a revolving jig; also in vertical joints and in cases where overhead welding is permitted, angular distortion being balanced out by welding the runs on either side simultaneously or alternately. To ensure that the two halves of the seam shall be as nearly as possible equal in size when completed, the portion of the cross section which is to be welded first must be made somewhat larger than the other in order to allow for scraping out and rewelding the root. If the lower portion of the seam is to be welded overhead, it is desirable to arrange the root of the weld in the lower third.

Butt welds should be arranged as nearly as possible at right angles to the axis of the girder in order to require a minimum of weld metal. In some cases, where joints in wide flanged girders have been welded with inclined butt welds, a great deal of contraction has occurred and part of the web plate has been torn away from the seam (Fig. 11), but if square butt welds are used these defects do not occur. Double sided stiffeners and cross girders are connected to the web by means of thin fillet welds. Double fillet welds, in the same way as X welds, allow of the angular distortion being reduced or compensated by attention to the sequence of welding. Apart from this angular effect, transverse and longitudinal shrinkage occurs. Long seams are usually begun in the middle and run towards both ends simultaneously. The intermittent method is less used in bridge work as the number of beginnings that have to be made is liable to cause defects. It is possible to reduce transverse stresses and shrinkage by heat treatment of the completed section of weld metal. If the artificial heating of the seam ceases on the completion of the welding operation, transverse shrinkage may occur over the whole length of the weld and this can only result in a shortening of the member; this presupposes that all parts of the member are free to move easily and without restraint.

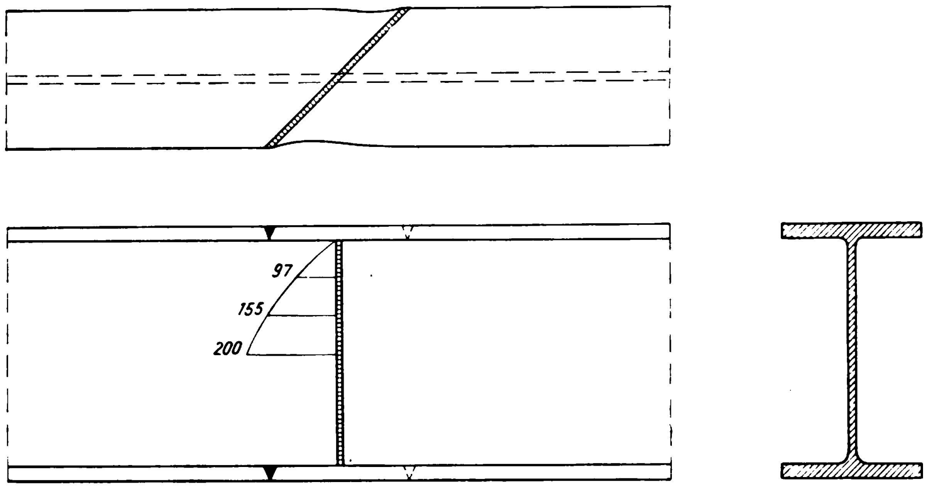

Fig. 11.

Apart from the shrinkages of the weld seam itself, expansion takes place in the adjacent zones on either side of the seam during the welding operation and is later followed by contraction. These effects cannot be mitigated by additional heating before and after the operation but all that can be done in this way is to equalise the stress. Unrestrained expansion during the process of pre-heating, and contraction afterwards, may be ensured by the use of moveable bearings.

The requirement of free movement for the work while being welded is easily secured in the workshop, and this being the case the web plate and flanges are welded together in long pieces, which are connected finally to form the girder. The device of welding the stiffeners onto the web plate before making the welds between the web and the flanges frees the latter from stress, whereas if this sequence is reversed the greater distortion of the web causes additional stresses in the seams. On the other hand, the accurate fitting of stiffeners serves to prevent cross shrinkage at the places where they are affixed, with the result that the flange is apt to assume an undulating shape, a defect which cannot be prevented by leaving the middle portion of the web plate to be inserted later, with pre-heating. If, however, the stiffeners are arranged with a certain amount of play, it is desirable to have available a number of packers of different thicknesses so as to be able to take up the slight changes in length wich result from the shrinkage. If there is a gap between the flange plate and a stiffener it is wrong to form a connection by welding this gap, as such a procedure would have the effect of drawing down the flange plate. When the girder is assembled the longitudinal contractions of the seam become apparent as a shortening of the work as a whole, and since the magnitude of the longitudinal shrinkage depends on the length of the seams it is customary to begin the weld at the middle and work towards either end. It is an advantage to connect both the flanges to the web plate simultaneously, or else to weld the top and bottom seams alternately, so as to prevent the web plate becoming curved towards one side.

In some types of bridge sections — for instance, in bridges where the decking is formed from welded on buckler plates, trough plates, or flat plates — weld seams are crowded together in the upper flange of the road girders, and since the result of the longitudinal contraction is to shorten the upper boom, bending of the girder takes place this effect must be allowed for in advance by giving an additional camber.

Continuous girders of large size and length require to be assembled on the site, which has hitherto been effected on rollers. This arrangement, however, has the disadvantage that any movement perpendicular to the rollers encounters great frictional resistance which hinders shrinkage in that direction. A method has, therefore, recently been adopted whereby the pieces of the girder are supported, while being welded, in such a way that they can move longitudinally and that each piece added to what is already in position can freely follow the shrinking movement. This arrangement necessitates the occasional use of overhead welds, but under the present conditions of the art of welding these offer no difficulty. The sequence in which the various seams of the girder joint are to be welded is determined by the rule that whatever seam offers the greatest resistance to the free movement erection of the work piece should be welded last.

Flange joints are usually arranged on either side of the web joints. The top and bottom welds, connecting the flanges to the web plate, are left open over a considerable length, so that shrinkages in the flanges and web plate may occur independently. The welding of both the flange seams is begun at the same time, and the gap at the root of the web seam is increased in such a way that after shrinkage in the first layers has taken place, it is still possible to ensure perfect penetration of the web seam. By alternately welding the flange and web seams it is possible to combat the accumulation of stresses, and to obtain a balance between compressive and tensile stresses. Thus the shrinkage of each successive layer of the flange seam causes compression in the layers already completed of the web seam, and in this way the tensile stresses set up by the shrinkage of the web seam are partially compensated. This process is repeated for each layer, so that finally the only stresses remaining are those which are produced by the final layers of the seams, in particular by a weld layer with a larger amount of weld metal. It is a common practice to weld first one-third of the flange seams, then two-thirds of the web plate seams, finally completing the flange seams and the web seams simultaneously ultimately forming the top and bottom neck seams between web and flanges.

IV. In the last few years a number of all-welded bridges have been built, characterised by the straightness and continuity of the girders and extreme simplicity of design. From the aesthetic point of view welded structures are superior to rivetted.

IIIa 2

Dynamic Stresses on Welded Steel Structures.

Dynamische Beanspruchungen bei geschweißten Stahl-konstruktionen.

Actions dynamiques sur les constructions soudées.

A. Goelzer,
Directeur de la Société Secrom, Paris.

The object of this contribution is to draw attention to a difficulty which, in the writer's opinion, arises in regard to the study of dynamic action on welded structures.

Generally speaking the effect of moving live loads on bridges and building frameworks is to bring into play forces of inertia. Such forces call for a "live" resistance, or resilience, in place of the usual kind of statical resistance which is exerted by materials. It may be observed that in the case of butt welded connections the resilience is always considerable, amounting to at least 8 kg/cm². In practice still higher vlues are obtained, being of the same order as the resilience of the parent metal, approximately 12 kg/cm². From this point of view it would appear, then, that the average resistance of a welded structure, taking full account of the presence of joints, is high enough to ensure that such a structure will behave well under live loads of the kind which may give rise to impact.

The writer has in fact found this to be true as regards the swing bridge at Brest, which, after having undergone strengthening operations, was subjected to tests under the direction of Mons. *Cavenel,* Ingénieur en Chef des Ponts et Chausseés, and Mons. *Lecomte,* Ingénieur des Ponts et Chausseés. These tests yielded very satisfactory results, it being found that after the strengthening work the vibration was considerably reduced.

For some time past a good deal of importance has also been attached to fatigue tests. It is known that if a solid — particularly steel — is subjected to forces which are repeated a great number of times fracture may occur even though the limiting resistance or even the elastic limit has not been reached. This fact obviously implies a serious danger due to dynamic action and the question is indisputably one which deserves the closest examination. The writer is of opinion, nevertheless, that this danger should not be exaggerated, for it is a fact that the majority of framed structures are not subjected to repeated loading of the kind that occurs in mechanical engineering.

A very complete investigation of this problem which has been carried out in France by Mons. *Dutilleul,* Ingénieur du Génie Maritime, indicates that lack of fatigue resistance in welds is always attributable to the presence of air bubbles, or in other words, to the porosity of the metal. In the present state of knowledge on the subject it appears, to the writer, dangerous to adopt as a criterion of quality of welding something which depends partly on chance, and he is of opinion that the resilience is the most important characteristic to consider.

In any case, whether it is resilience or fatigue resistance that is the most important element, there remains the over-riding necessity of ensuring that the weld bead shall be given a shape which will not tend notably to increase the

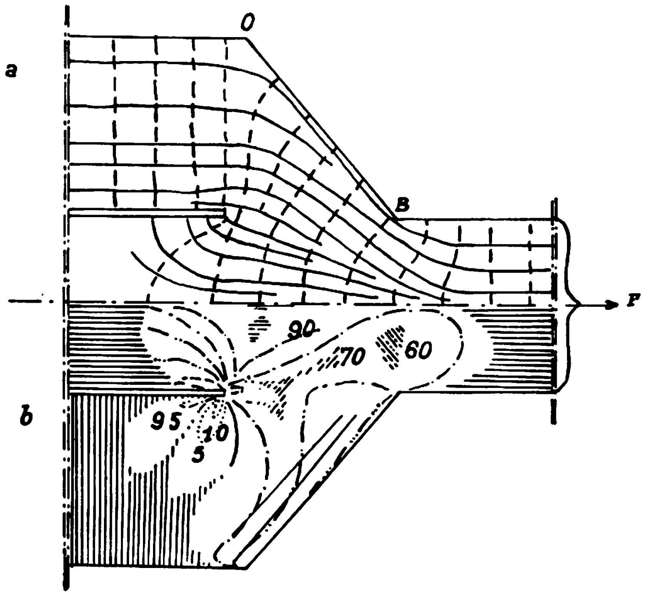

Fig. 1.

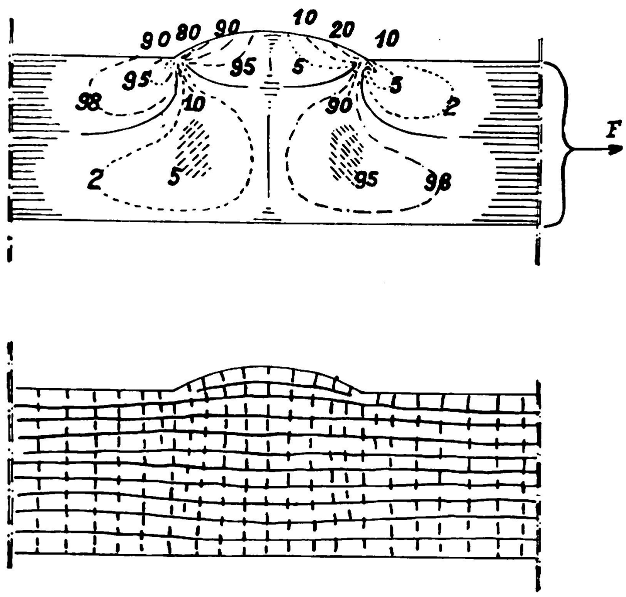

Fig. 2.

risk of rupture. On this subject, as Messrs. *Kommerell* and *Graf* have shown, valuable data can be inferred from the study of lines of principal stress under static loads.

The two illustrations here given go to confirm this argument, representing as they do the lines of force in a transverse weld and in a butt weld (Figs. 1 and 2). These were obtained by the use of polarised light applied to a celluloid model shaped and stressed in the same way as the welded piece. The photographs show very clearly that it is important to minimise any disturbance of the flow of the lines of force. For instance, in the case of the butt weld it may be observed that an excess of thickness may be harmful, in that it perceptibly increases the amount of strain suffered at one of the faces of the specimen.

IIIa 3

Questions for Discussion on Fundamental Relationships and Principles Governing the Fatigue Strengths of Welded Connections.

Diskussionsfragen über Grundbeziehungen und Begriffsfestsetzungen für die Dauerfestigkeit geschweißter Stabverbindungen.

Thèmes de discussion concernant les relations fondamentales et la détermination des notions se rapportant à la résistance à la fatigue des assemblages soudés.

Dr. Ing. W. Gehler,
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During the last five years numerous experiments on fatigue have been carried out in the German laboratories for testing materials, with a view to the formulation of official regulations for welded connections of structural members. The interpretation of these experiments has given rise to a series of questions, some of which it has been possible to clear up, while others constitute problems regarding which an international exchange of ideas is desirable and which demand further research.

1) *Representation of the fundamental dimensions force, space and time (Fig. 1).*

a) The ordinary force-space plane X—Y (or stress-strain plane) serves to represent the results obtained in the ordinary statical breaking test, which lies at the basis of the theory of strength and elasticity. The effect of the time factor on the breaking test is usually ignored, but may be recognised in the fact that if the experiment is carried out more quickly the usual line 1 in Fig. 1 merges into line 2, or in the case of impact tests may even merge into line 3.

b) If the third axis of coordinates Z is adopted as the time axis, then the Y—Z plane shows the transition to the region of vibration, or to time-strength relationships, and the result of fatigue tests may be represented in it by a fatigue-time line (known as the *Wöhler* line).[1] Here the abscissae z represent the duration of the experiment, though not according to the usual time scale, for it is expressed by the number of alternations of load (for instance 2 million alter-

[1] *Wöhler:* Zeitschrift für Bauwesen, Berlin 1860, 1863, 1866 and 1870.

nations of 4 seconds each, or 8 million seconds in all). If time is measured in this way, by the number of similar vibrations instead of by the clock, the *Wöhler* line shows how many vibrations can be withstood by the specimen under a given stress. If the experiment continues long enough this line finally becomes parallel to the Z axis at a distance σ_D where it represents the fatigue stress *(Dauerfestigkeit)* that is to say, the amount of stress that can be supported indefinitely.[2]

c) The range of vibration covers a system of coordinates X′ Y′ Z′ which has been displaced parallel to the XYZ system, the stresses being again indicated by the ordinates y′ measured along the direction of the load axis. In the Y′ Z′ plane (the load-time plane) it is possible to visualise the following three fundamental concepts: I) alternating strength for equal positive and negative stresses (*Wechselfestigkeit* σ_W), II) fatigue strength for stresses pulsating between zero and a maximum (*Ursprungsfestigkeit* σ_U), and III) fatigue range (*Schwellfestigkeit* $\sigma_{D\max}$), which is the fatigue strength subject to a pre-existing stress σ_m (In reference to I) the limiting values of the stress wave are $\sigma_o = +\sigma_w$ and $\sigma_u = -\sigma_w$ hence $\sigma_m = 0$; in reference to II) they are $\sigma_o = \sigma_U$ and $\sigma_u = 0$, hence $\sigma_m = \frac{1}{2} \cdot \sigma_U$).

d) A plane X″ of X′ rotated about the Y-axis may serve for the representation of any other function of fatigue strength, the stresses σ_D always being measured by the ordinates y′. In this way, for instance, it is possible to indicate the relations of the stresses to the depths of notches in notched bars, or to describe the y — method of the Reichsbahn, wherein $x' = \sigma_{min} : \sigma_{max}$ is introduced, or to indicate the relationship to statical pre-stressing (*Haigh*'s construction).

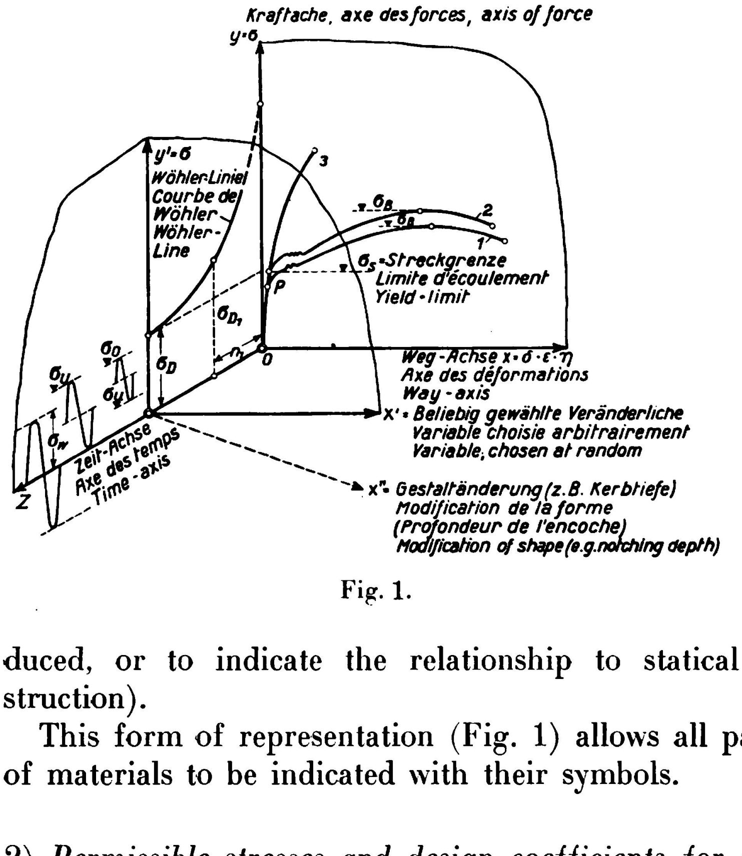

Fig. 1.

This form of representation (Fig. 1) allows all parts of the theory of strength of materials to be indicated with their symbols.

2) *Permissible stresses and design coefficients for steel bridges.*

A) *Rivetted railway bridges.*[3]

Whereas, under stationary or statical loading, safety is determined by the proportions between the stress σ_s which corresponds to the yield point, and the

[2] The first two of the questions for discussion enumerated in Section 8 arise here.

[3] The tern "railway bridges" is used to denote bridges which carry railway tracks in contradistinction to roadways.

stress σ_{zul} which arises under live load (so that $v = \sigma_s : \sigma_{zul}$), under conditions of alternating load safety is determined by the *number of alternations*, and this is a statistical problem.

Case 1. Side members of a trussed main girder in a rivetted railway bridge (Fig. 2).

On an old trussed bridge of 39 m span[4] a tensile stress $\sigma_{max} = + 215$ kg/cm² and a compressive stress $\sigma_{min} = - 70$ kg/cm² were recorded graphically during the passage of two test vehicles of $4 \times 8 = 32$ tons weight, and the curve of stress was determined graphically as in Fig. 2, corresponding to the total influence line of the test crane. If, during the passage of a locomotive the designed

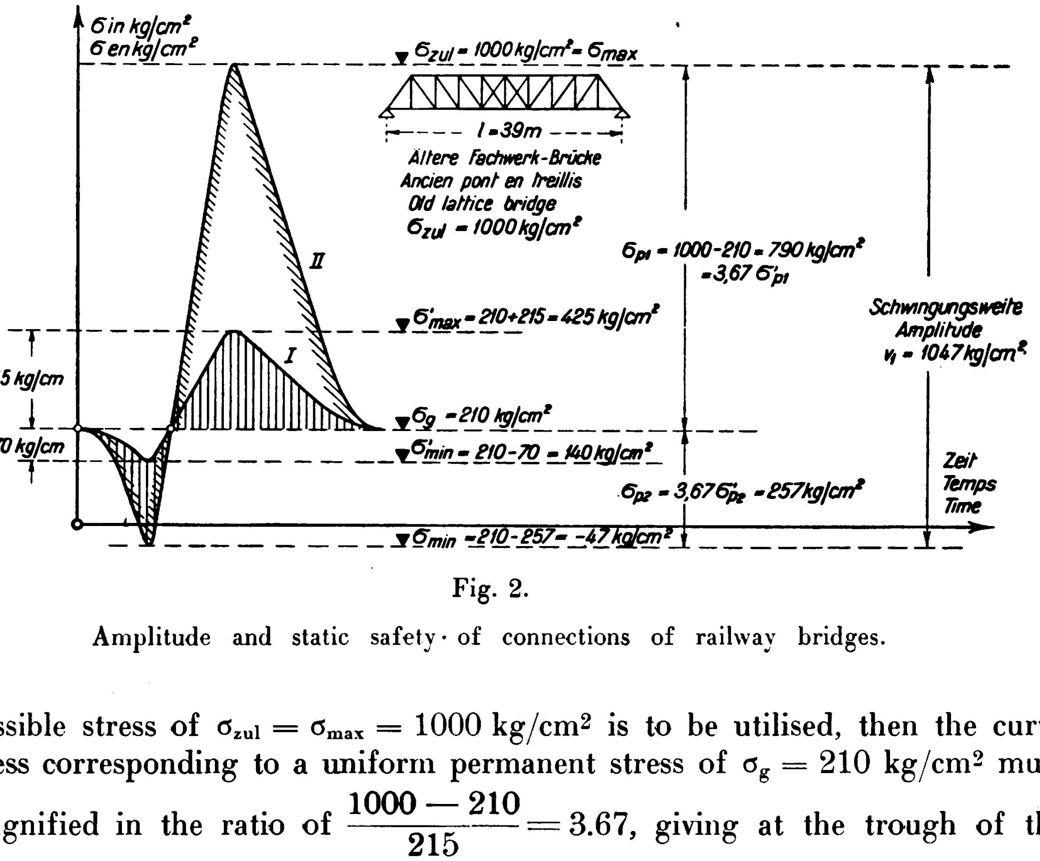

Fig. 2.

Amplitude and static safety of connections of railway bridges.

permissible stress of $\sigma_{zul} = \sigma_{max} = 1000$ kg/cm² is to be utilised, then the curve of stress corresponding to a uniform permanent stress of $\sigma_g = 210$ kg/cm² must be magnified in the ratio of $\frac{1000 - 210}{215} = 3.67$, giving at the trough of the wave $70 \times 3.67 = 257$ kg/cm², and $\sigma_{min} = 210 - 257 = - 47$ kg/cm² (compression). The total amplitude then works out at

$$v_1 = 1000 + 47 = 1047 \text{ kg/cm}^2 \doteq 10.5 \text{ kg/cm}^2. \quad (1)$$

If the problem now arises of determining the fatigue strength σ_{D1} of this member and its connections experimentally, the amplitude must in the same way be fixed at $w_1 = v_1 = 10.5$ kg/cm² in the experiment. If only a small portion of the stress curve for the tensile member in question falls within the compression zone, the fatigue test may properly be based upon the pulsating strength *(Ur-*

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4 *W. Gehler:* Nebenspannungen eiserner Fachwerkbrücken, p. 67 (Wilh. Ernst & Sohn, Berlin 1910).

sprungsfestigkeit) ($\sigma_{min} = 0$; $\sigma_{max} = 10.5$ kg/cm²). For instance, if the fatigue test shows breakage at $n_{D1} = 2000000$ changes of load,[5] and if the bridge is traversed daily by $n_1 = 25$ trains, then this number becomes n_{D1}, and failure need not be apprehended until at the earliest 80000 days or 220 years have elapsed (but under tramway traffic with $n_T = 250$ vehicles per day failure might be expected in only 22 years). Thus the criterion of safety may be expressed in the form of the life of the bridge in days and we have the relationship[6]

$$\nu_T = n_D : n_T \tag{2}$$

A fatigue experiment of this kind, however, does not yield a true picture of the situation, because it is carried through without interruption, whereas in the actual structure long pauses intervene, especially at night, and in these pauses it is conceivable that a recovery of the material may take place. Even if no influence due to rest pauses has hitherto been disclosed by the experiments as regards the fatigue resistance of the materials themselves, it may well be possible that the conditions in regard to rivetted or welded *connections* are more favourable in this respect.[7]

Case 2. Boom of a continuous solid-webbed or openwebbed main girder.

This example shows the necessity of separating, in the longitudinal section of the girder, the zone in which the calculated limiting stresses σ_{max} and σ_{min} have the same sign from the alternating zone in which they have different signs. Thus according to the ratio

$$\xi = \frac{\min S}{\max S} \text{ or } \frac{\min M}{\max M} \tag{3}$$

of the statical load S and moment M in the bars, a vibration coefficient

$$\gamma = \frac{\sigma_{zul}}{\sigma_{D\,zul}} > 1 \tag{4}$$

$\left(\text{corresponding to the buckling coefficient } \omega = \frac{\sigma_{zul}}{\sigma_{D\,zul}}\right)$ is to be introduced, because in the fatigue experiments the alternating strength, the pulsating strength and the fatigue range have different values for St. 37 and St. 52. In this way the different $\gamma - \xi$ lines of the Reichsbahn are obtained (B.E., Berechnungsgrundlagen für deutsche Eisenbahnbrücken, Section 36, Table 17). In a similar way to the assumption of $\frac{\omega \cdot S}{F} \leqq \sigma_{zul}$ for buckling, the stress is here to be taken as

$$\sigma_I = \frac{\gamma \cdot \max S}{F} \leqq \sigma_{zul} \tag{5}$$

and the calculation may then be performed in exactly the same way as for members under purely statical loading.[8]

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[5] Compare first question for discussion.

[6] Compare fourth question for discussion.

[7] Here the third question for discussion arises.

[8] *Kommerell:* Erläuterungen zu den Vorschriften für geschweißte Stahlbauten, Part II, page 39 (Wilhelm Ernst & Sohn, Berlin 1936).

Case 3. Connections between longitudinal and cross girders.

It is a matter of experience that rivets at these connections easily work loose in service, and in the new regulations of the Reichsbahn (B.E., Section 46) it has been sought to promote safety not only by making the design assumptions more severe (increased thrust max $A' = 1.2\ (A_g + \varphi A_p)$ and increased bending moment in St. 52 compared with St. 37), but also by the requirement of special constructional precautions (such as provision, in every case, of a plate running through on top). The reduced span of the longitudinal girder is also safeguarded by adopting a higher value of the impact coefficient φ, such as for instance $\varphi = 1.6$ under permanent way with sleepers and $l = 5.0$ m instead of $\varphi = 1.4$ for the main girders of medium span. The only way to estimate the true magnitude and effect of the stress variations in this complicated special case, where the distribution of loading is influenced by the superstructure, and to compare it with the results of fatigue experiments on similar types of connection, would be to carry out exact measurements on actual bridges — a problem which still awaits research.

B) Welded railway bridges.

Account is taken of live loading effects in the following ways:

a) By placing stationary train loads in unfavourable positions and plotting influence lines.

b) Impact coefficients of $\varphi \geqq 1$ are adopted (wherein $S = S_g + \varphi \cdot S_p$ or $M = M_g + \varphi\ M_p$) in order to allow for the effect of impact and vibration through the movement of the loads, by comparison with stationary loads (for instance as a result of driving wheel action, rail joints, etc.). Such effects tend to increase the statical deflection. (This is destined to be a principal problem of bridge investigation in the future).

c) The *vibration coefficient* $\gamma \geqq 1$ is expressed as a function of the calculated statical limits min S and max S in order to allow for the difference in fatigue effects on the structural member under alternating and pulsating loads, and apart from this, different values are used for St. 37 and St. 52; also different values according as the traffic is heavy or light ($n_T = 25$ or $n_1 \geqq 25$ trains per day).

d) The *design reduction coefficient* $\alpha \geqq 1$. Whereas the coefficients γ may be fundamentally the same for rivetted and welded bridges the permissible stresses for welded railway bridges have been still further reduced in accordance with the German fatigue experiments,[9, 10] becoming (see equation 5):

$$\sigma'_I = \frac{\sigma_I}{\alpha} = \frac{\gamma \cdot \max S}{\alpha \cdot F} \leqq \sigma_{zul} \quad \text{or} \quad \frac{\gamma}{\alpha} \cdot \frac{\max M}{W} \leqq \sigma_{zul}, \tag{6}$$

wherein the design factor α is given a different value according to the form of seam (whether a butt weld or a fillet weld) and according to the quality of

[9]) Dauerfestigkeitsversuche mit Schweißverbindungen, 1935, VDI-Verlag, Berlin. Joint report of Staatl. Materialprüfungsamt Berlin-Dahlem and Versuchs- und Materialprüfungsamt Dresden. By *K. Memmler, G. Bierett* and *W. Gehler*.

[10] See footnote 8, *Kommerell*, page 44.

workmanship (e. g., whether the root of the seam has been re-welded or not, and whether the finished seams have been improved by further working). Reduction coefficients of this kind are already in use for welded building frames (DIN 4100, Section 5) wherein, for instance, butt welds may be stressed in tension to $\rho_{zul} = 0.75\ \sigma_{zul}$, hence $\alpha = 0.75$.

C) *Rivetted and welded road bridges.*

Road bridges are much less frequently exposed to sustained alternating loading than is the case with railway bridges, and since, moreover, the German loading assumptions (DIN 1073) already ensure ample safety as regards weight and

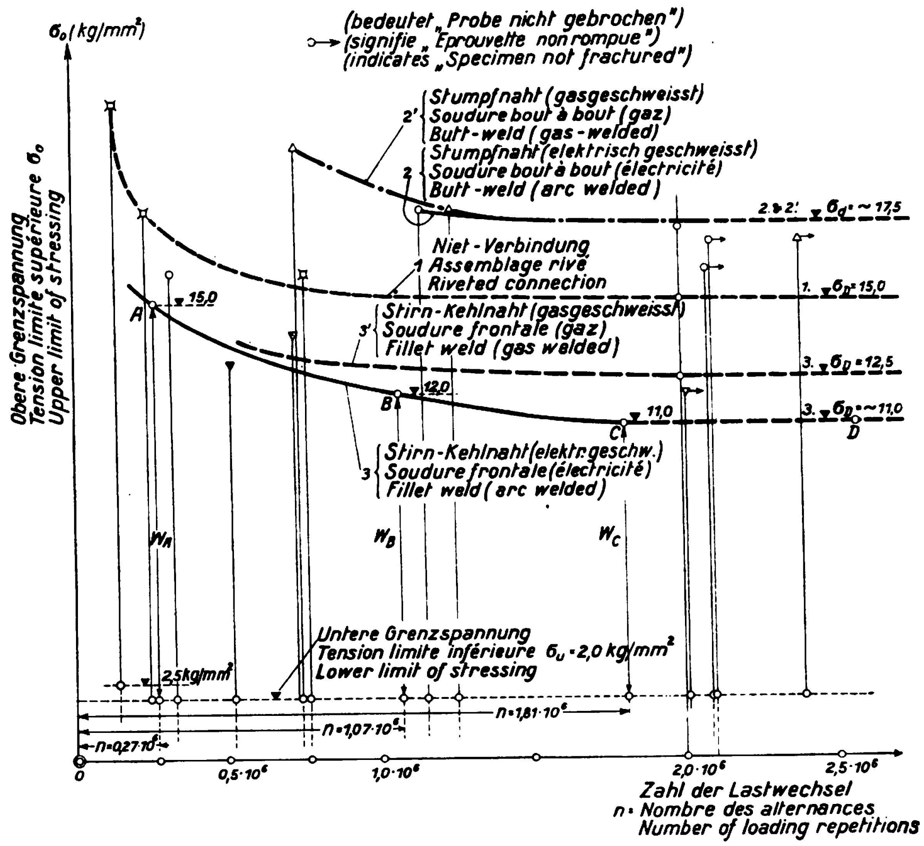

Fig. 3.

Wöhler lines according to Report of Board of Administrators with welded and riveted connections of members stressed in Tension.

traffic density of the loads, these bridges are nearly always regarded as being statically loaded. Consequently the vibration coefficient may be omitted from among the four effects enumerated under B) above and γ may be put equal to unity, while retaining the impact coefficient φ, and also certain design reduction coefficients α in the case of welded road bridges.

3) *The limiting stress-time curve (Wöhler's curve)* (Fig. 3).

Since the fatigue strength σ_D depends on a number of variables (such as n, σ_o, σ_u or σ_m) it is desirable to represent these in different planes with the axes Z, X′ and X″ respectively as shown in Fig. 1. The first requirement is the recording of experimental results in the form known as the *Wöhler* curve. Suppose, for instance, that the stress σ_D is to be determined in the pulsator by means of tensile tests for electrically welded side fillet welds[11] (Fig. 3, Line 3). An upper limiting stress is first fixed arbitrarily at, for instance, $\sigma_o = 15$ kg/mm² with a lower limiting stress of $\sigma_u = 2.0$ kg/mm² and it is found that breakage takes place at n = 270000 changes of load (Point A). On a second attempt

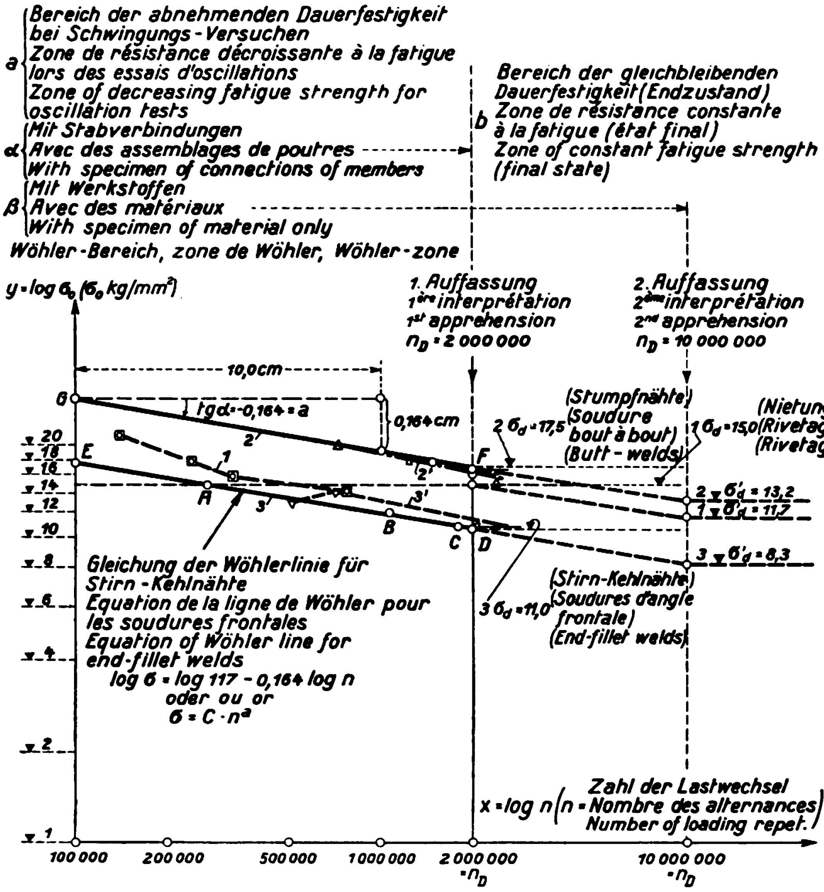

Fig. 4.

Zone of decreasing fatigue strength (Wöhler zone) of the stress-time curve (plotted logarithmically) for riveted and welded connections.

being made with $\sigma_o = 12$ kg/mm² and the same value of $\sigma_u = 2.0$ kg/mm² there is obtained n = 1.07 million (Point B) and finally in a third experiment with $\sigma_o = 11.0$ kg/mm² there is obtained n = 1.81 million (Point C). Since the portion CD of the line ABC is already approximate horizontal, the final value of the fatigue strength may be assumed at $\sigma_D = \lim \sigma_o$ = approximately 11 kg/mm².[12]

This experiment may now be plotted as in Fig. 4 with $y = \log \sigma_o$ as ordinates and $x = \log n$ as abscisse, the logarithmic scale being adopted along both coordinate axes *(and not only along the X axis, as is commonly done)*. It is then

[11] See footnote 9.

[12] First question for discussion.

found that the line ABC approximates closely enough to a straight line ED, which with the parallel line GF indicates the trend of direction of the remaining experimental lines. The equation for the straight line ABC is as follows:

$$\log \sigma = \log 117 - 0.164 \log n \tag{7a}$$

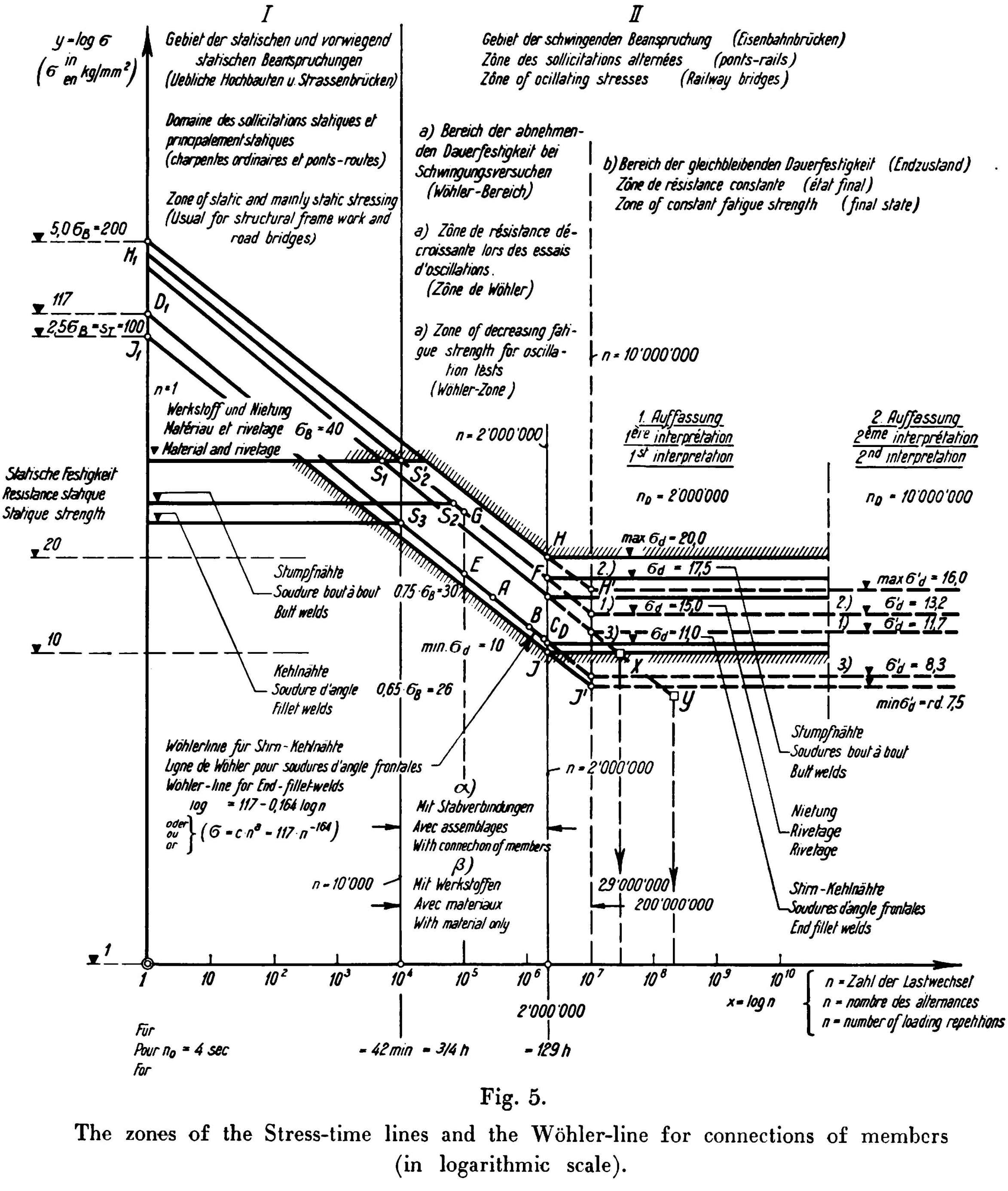

Fig. 5.

The zones of the Stress-time lines and the Wöhler-line for connections of members (in logarithmic scale).

and this corresponds to the exponential curve[13] as represented in Fig. 3:

$$\sigma = C \cdot n^a \tag{7b}$$

[13] See also Moore, Am. Soc. Test. Mat. 1922, p. 266 and Basquin, ditto, 1910, p. 625.

wherein $a = \tan \alpha$ 0.164 gives the slope of the lines and $C = 117$ kg/mm^2 gives the value of σ for $n = 1$.

In Fig. 5 the whole of the limiting stress-time curve is shown in the same logarithmic form. For *the region of falling fatigue strength,* which will here be designated the *Wöhler* region, we have thus set limits at the respective points $n = 10000$ and $n = 2000000$. On the left and right these connect with other horizontal portions, so that the limiting stress-time curve when represented in this logarithmic form consists of straight lines with two bends. The line DE cuts the coordinate axis ($n = 1$) with the ordinate $C = 117$ kg/mm^2. If, now, a line is drawn through the point J with $\sigma_D = 10$ kg/mm^2 and $n = 2000000$, and a line through the point J_1 to the axis of coordinates with $\sigma_D = 100$ kg/mm^2, and if a further line HH_1 is drawn parallel to J_1 with the point H corresponding to the value $\sigma_D = 20$ kg/mm^2, the result is to enclose a figure which contains practically the same values as are found in fatigue experiments on connections of bars.

The stress-time curve may then be divided up as follows. Firstly there is the region of statical or mainly statical stresses, such as occur in the usual forms of building frames and in road bridges. Secondly there is the region of alternating stresses, such as occur in railway bridges — further divisible into a portion where the fatigue strength is falling away as in vibration experiments (the *Wöhler* region) and a portion where the fatigue strength remains constant and may be looked upon as a final condition (IIa and IIb).

The sub-division into these two portions IIa and IIb is in itself arbitrary, and is the subject of Question 1 for discussion.[14] The knowledge hitherto available from fatigue experiments carried out on *connections* between structural members has led to the adoption of $n_D = 2000000$ (first assumption). If, however, as is usual in testing *materials,* $n_D = 10\,000000$ is substituted (second assumption), then the rectilinear projection of the lines in the *Wöhler* region (for instance, as far as the point H′ and J′ in Fig. 5) to correspond with rivetted practice, would give $\sigma_D = 11.7$ kg/mm^2 instead of 15 kg/mm^2 (see points V and W). The fact that rivetted railway bridges have given good performance under railway traffic when designed with $\sigma_{zul} = 10$ kg/mm^2 would then be difficult to reconcile with experimental results.

The relation to the fatigue strength of $\sigma_D = 15$ kg/mm^2 as found for rivetted connections in the fatigue experiments, to the permissible stress $\sigma_{zul} = 14$ kg/mm^2 in rivetted railway bridges, is

$$\nu_w = \frac{\sigma_D}{\sigma_{zul}} = \frac{15}{14} = 1.07$$

This affords a further margin of safety which, although small, may be relied upon to compensate for any possible lack of uniformity and quality of the material or other inaccuracies in execution.

Since in the testing of materials for $n > 10000000$ the stress strain line was assumed to be horizontal (Phase IIb) it is surprising that in the experiments carried out at Dresden the fatigue failure of a controlled specimen should have

[14] See Section 5 (First question for discussion).

taken place after 29000000 changes of load (see point X in Fig. 5) while in the testing machine at Berlin-Dahlem the fatigue failure at a rivet hole of a truss member should not have taken place until after 200000000 changes of load (see point Y). The second point for discussion is whether values of this order ($n > 10000000$) have been observed elsewhere, either in experiments or in railway service.[15]

In Fig. 6 the *Wöhler* line is again indicated without distortion of scale at the points V, W, X and Y. Referring to the fatigue test on structural connections, if the line had been terminated at $n = 10000000$ changes of load (point W) instead of at $n = 2000000$ (point V), then a fatigue strength approximately 20 % lower would have been obtained. In the exceptionally long-delayed fatigue

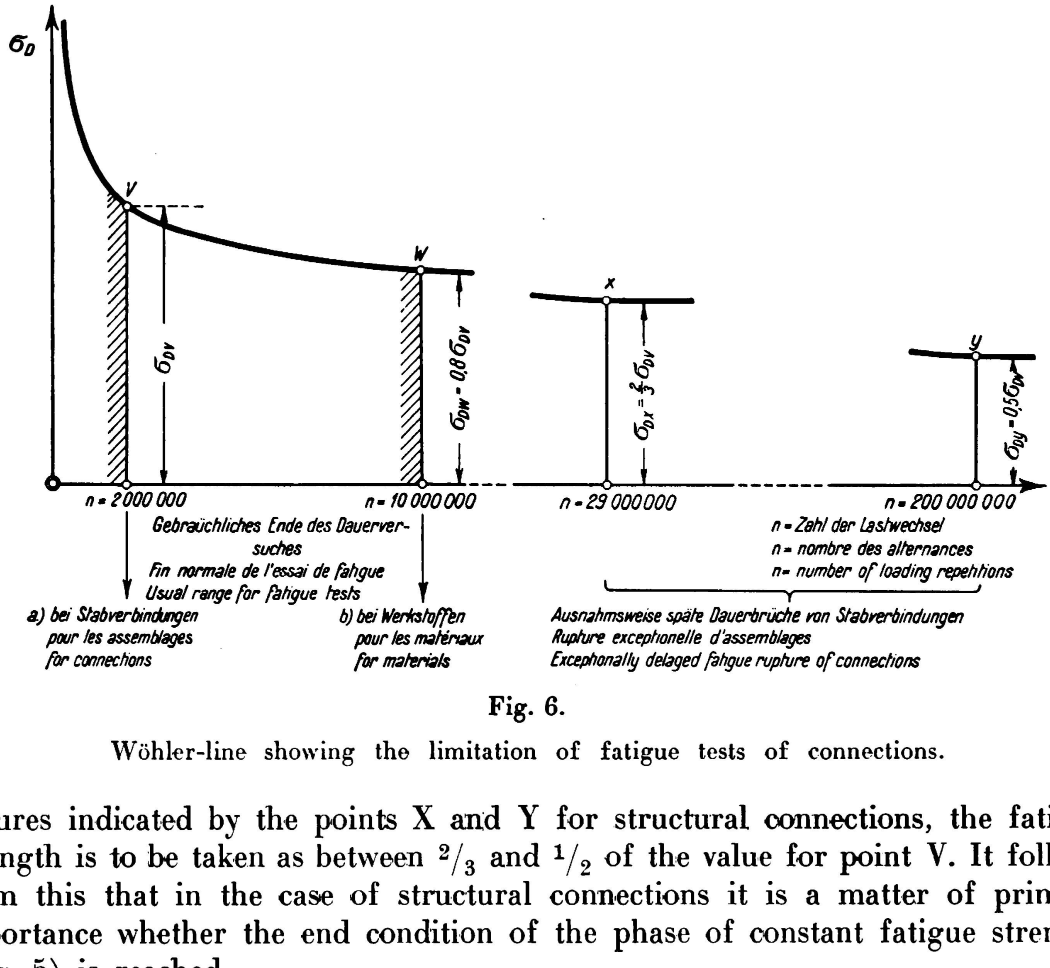

Fig. 6.

Wöhler-line showing the limitation of fatigue tests of connections.

failures indicated by the points X and Y for structural connections, the fatigue strength is to be taken as between $^2/_3$ and $^1/_2$ of the value for point V. It follows from this that in the case of structural connections it is a matter of primary importance whether the end condition of the phase of constant fatigue strength (Fig. 5) is reached.

By adopting this logarithmic presentation of the stress-time line, the whole field of statical and fatigue tests results may be incorporated in the stress-time plane. It should also be noticed that the ordinate of the point J_1, belonging to the lower limiting line JJ_1, corresponds approximately to the critical strength

$$s_T = 2{,}5 \cdot \sigma_B = 2{,}5 \times 40 = 100 \text{ kg/mm}^2 \tag{8}$$

and that the intersection points S_1 and S_2 also indicate a certain uniformity in their relationship to the horizontal lines of the statical region and the inclined

[15] See Section 5 (Second question for discussion).

lines of the *Wöhler* region. The test results obtained in the statical and fatigue tests would agree much better if stresses for butt welds were to be taken in future as 1.0 σ_{zul} instead of as 0.75 σ_{zul}; that is to say, if the points S_2 and S'_2 were to be shifted.

Physically, C in Equation (7b) may be regarded as a *coefficient of cohesion* (compare Equation 8). A meaning for the other coefficient *a* may then be inferred from the equation (7b) as follows:

$$y' = \frac{d\sigma}{dn} = c \cdot a \cdot n^{a-1} = 117 \cdot 0.164 \cdot n^{-0.836} = \sim \frac{19.2}{n}, \tag{9}$$

which for a first approximation corresponds to a rectangular hyperbola. In the *Wöhler* region the y′ line at first falls steeply downwards and finally it runs parallel to the axis of the abscissae (Fig. 3). If the ordinates $\sigma \left(\frac{kg \cdot mm}{mm^3} = \frac{kg}{mm^2}\right)$ be regardet as representing specific energy, or loading per unit volume of 1 mm^3, then the ordinates give a measure of the efficiency (work/time), and the y′ line represents the drop in this efficiency, or the fatigue experienced during the experiment. According to Fig. 5 the trend of this line is similar for the structural connections investigated and for the *Wöhler* region. The value of *a* may thus be designated as a fatigue coefficient ($a = 0$ in region I and IIb of Fig. 5).

4) *Comparison between the stresses actually arising in the structure according to statical calculations, and the stresses occurring in the test bars.*

Since the experiment has to be arranged as simply as possible it is confined to sinussoidal waves rising and falling between the upper and lower limits of σ_o and σ_u on either side of the average stress σ_m. Actually however — as indicated in Fig. 2 — the range of live load stress above and below the dead load stress σ_g is usually very different; in side members, for instance, being σ_{p1} above $> \sigma_{p2}$ below. In the boom members of girders σ_{p2} may even be equal to zero. Hence the experiment differs from reality not only as regards the shape of the waves, but also as regards their lack of symmetry in the different amplitudes (σ_{p1}) above and (σ_{p2}) below, Unfortunately, again, the effect of this discrepancy has not yet been investigated, and this suggests a further field for research.

5) *Questions for discussion.*

Question 1) Is it expedient to refer fatigue tests on connections of bars to a number of changes of load $n_D = 2000000$ instead of to $n_D = 10000000$ as is customary in the testing of materials? (Compare footnotes 2, 5, 12 and 14; also Figs. 5 and 8, points S and W).

Question 2) Have the exceptionally long delayed fatigue breakages, after 29 and 200 million changes of load, as observed in the German experiments, been confirmed elsewhere either in fatigue experiments on structural connections or in actual railway service? (Compare footnote 2 and 15; also Figs. 5 and 6, points X and Y).

Question 3) Is there any experimental evidence that a favourable effect on the fatigue strengths of structural connections may be exercised by rest pauses? (Compare footnote 7).

Question 4) Seeing that rivetted connections, which form the basis for investigating the behaviour of welded connections, give an average fatigue strength of $\sigma_D = 15\ kg/mm^2$ with a permissible stress (including impact allowance) of $\sigma_{zul} = 14\ kg/mm^2$, may the criterion for the safety of railway bridges properly be taken as the proportion between the number of changes of stress applied in the experiments and the number of trains per day? Or in other words, is the question answered by stating the life of the bridge in years $v_T = n_D : n_T$ making use of a *statistical* conception of safety? Can any other suitable suggestions be put forward for designating safety?

III a 4

Characteristic Features of Welding.

Charakteristische Merkmale der Schweißung.

Caractéristiques propres à la soudure.

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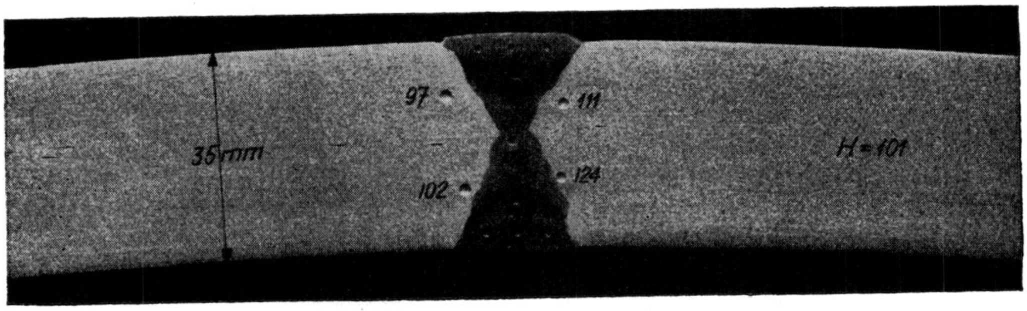
Professor an der Eidg. Techn. Hochschule und Direktionspräsident der Eidg. Materialprüfungs- und Versuchsanstalt für Industrie, Bauwesen und Gewerbe, Zürich.

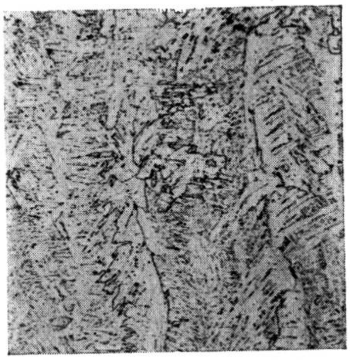
As regards the strength and deformation properties, the quality of the structure of the weld material, including that of the transition zone, is fundamental. Weld metal is in fact a form of steel casting; its structure is heterogeneous and anisotropic — Fig. 1 —, and the theory of constant strain-energy of deformation holds good only if account is taken of the anisotropic condition. In practice, no guarantee can be given that *weld material* is *free from pores* and *slags*. *Shrinkage-cracks* in the outer surfaces and in the interior of the weld material are, as a matter of fact, rare occurrences, but occasionally they may be present — Fig. 2 —. Pores, slag inclusions and cracks must be considered equivalent to a *mechanical imperfection*. *Thermal influences* give rise, on the surface, to the structure known by the name of Widmannstaetten with transcrystallization — Fig. 3 — and in the transition zone to the formation of sorbite, troostite — Fig. 1 — and — in steels containing less than 0.15 % carbon — also martensite — Fig. 4 — which on account of its brittleness promotes the formation of cracks. So far as the material itself is concerned, welding *cannot* therefore be considered equivalent to riveting — Fig. 5 —.

By metallurgical means such as proper choice of the electrodes, correct procedure with pre-heating of the steel when necessary — Fig. 6 —, heat treatment without stress (up to the lower transition-temperature at the most) or annealing (beyond the upper transition-temperature) combined with suitable precautions in design — i. e. reduction of thermal stresses — Fig. 7 —, proper choice of type of connection — Fig. 8 — and shape — Fig. 9 — the mechanical-caracteristics of the welded connection may be made to approximate to those of the riveted connection.

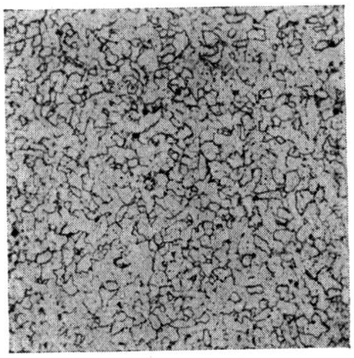
The series of tests and precautions is completed by examination of the welders based on the results of mechanical and deformation tests, either on plates specially welded both in the normal and overhead position or on specimens of suitable shape (round or oval) removed from work carried out in actual practice, combined with X-ray examinations.



$\beta_z \cong 38$ kg/mm² $\sigma_u \cong 16$ kg/mm²

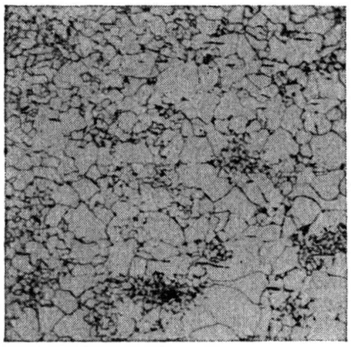


Coarse Widmannstaetten structure in weld metal.



Fine, normalised structure in weld metal.

Thermally altered structure, ferrite and sorbite, transition zone.



Thermally altered structure ferrite and degenerated perlite, transition zone.

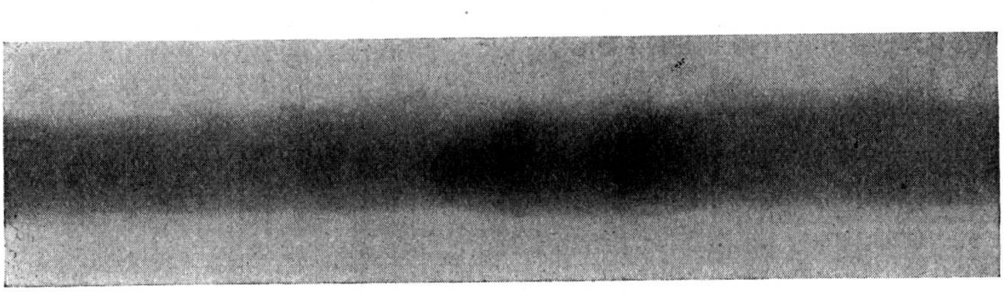
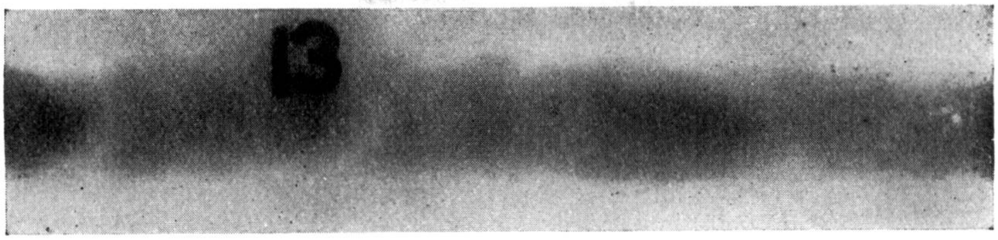
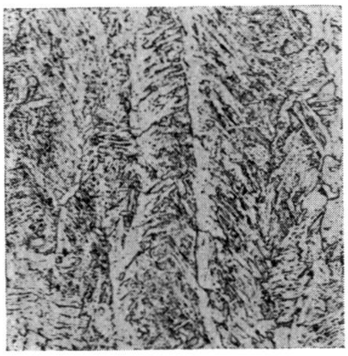


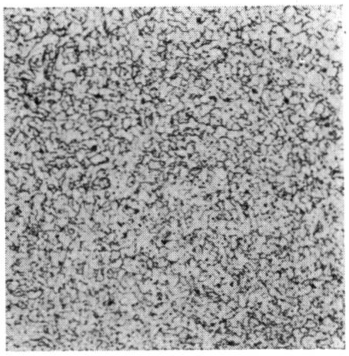
Fig. 1.

Arc weld free from defects, in normal structural steel. Heterogeneity of the weld metal structure.

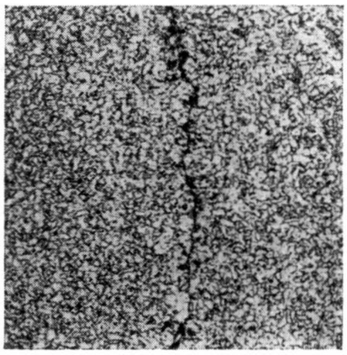




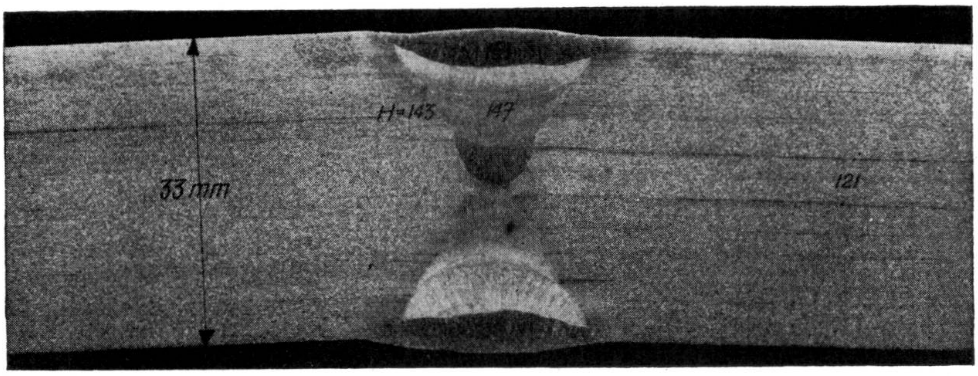
Coarse Widmannstaetten structure, weld metal, last layer.



Fine, normalised change in structure, welt metal.



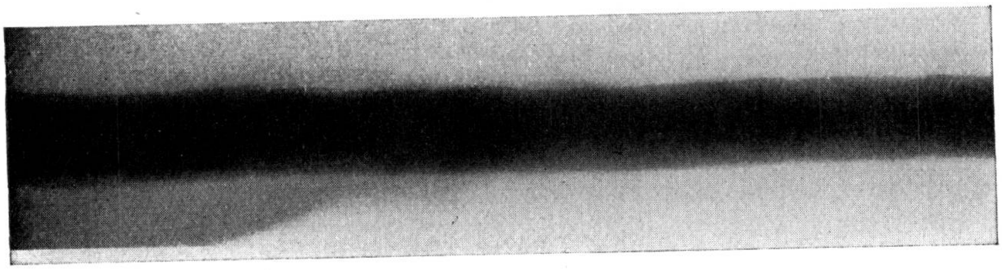
Fine, normalised change in structure with microscopic crack, weld metal.

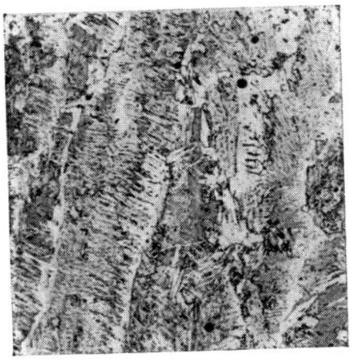


$\beta_z \cong \mathbf{44}$ kg/mm² $\qquad \sigma_u \cong 17$ kg/mm² $\qquad \frac{1}{2}\,\sigma_w \cong 27$ kg/mm²

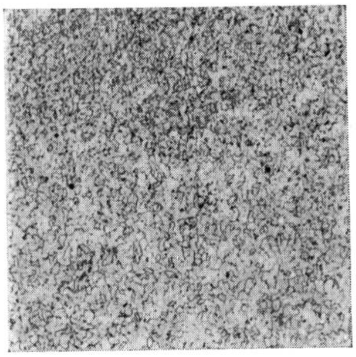
Fig. 2.

Arc weld of superior quality in steel containing 0.25 % C. Microscopic cracks in weld metal.

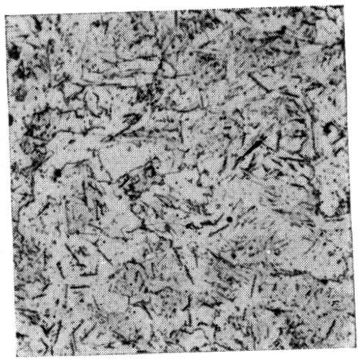




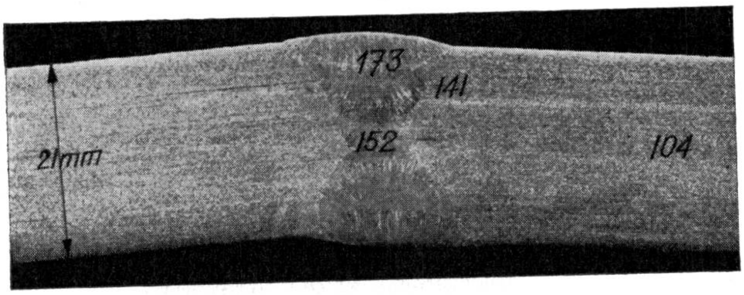
Widmannstaetten structure, weld metal.



Fine alteration in structure, weld metal.



Local enrichment of nitride inclusions, weld metal.



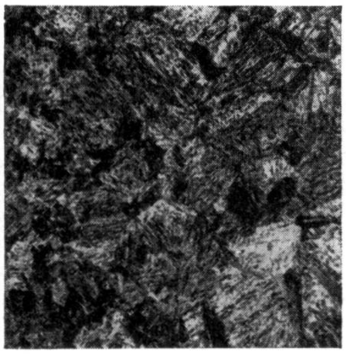
$\beta_z \cong 38\ \mathrm{kg/mm^2}$ $\sigma_u \cong 18\ \mathrm{kg/mm^2}$ $\frac{1}{2}\sigma_w \cong 26\ \mathrm{kg/mm^2}$

Fig. 3.

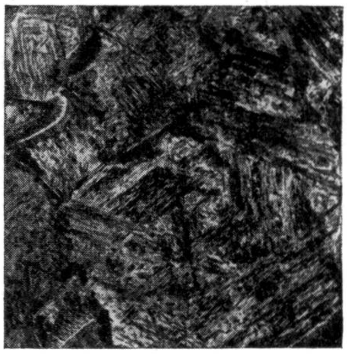
Arc weld free from defects, in normal structural steel. Widmanstaetten structure with transcrystallisation.

Siemens-Martin steel with 0.20—0.25 % C.

Temperature: — 10° C.
Martensite with traces of troostite due to quenching effect.



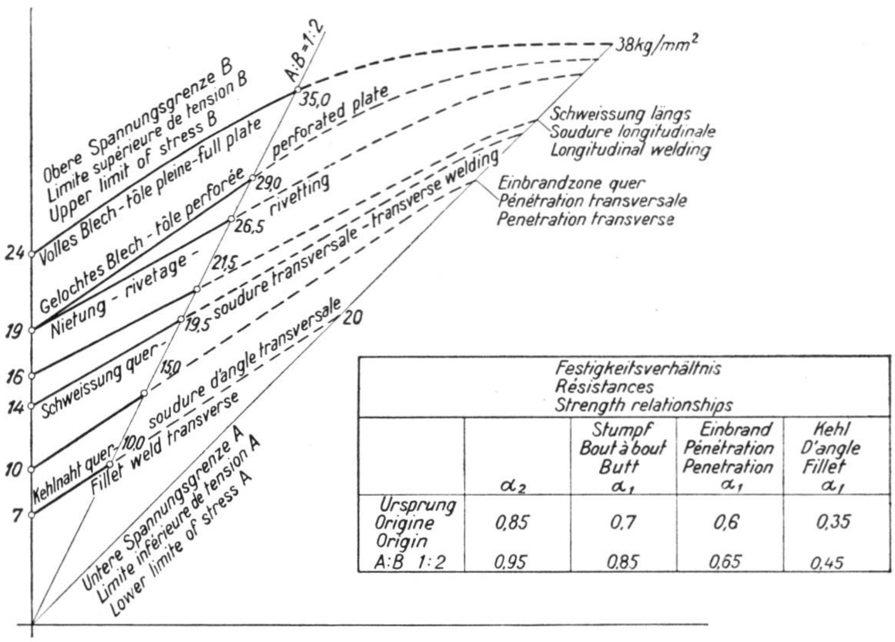
Temperature: 25° C.
Martensite with troostite due to quenching effect.



Temperature: 50° C.
Martensite with little troostite, traces of ferrite.

Fig. 4.

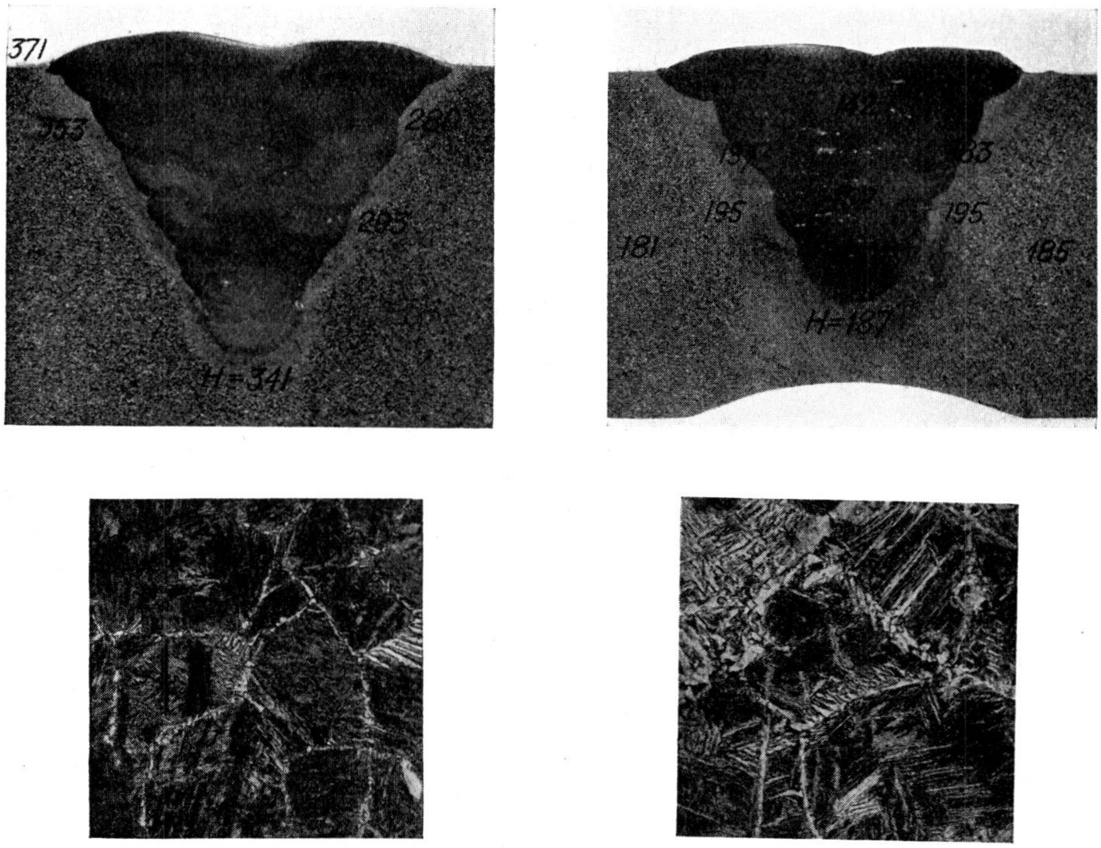
Formation of Martensite in the transition zone immediately at the junction of the weld metal with Siemens-Martin steel plate.
Temperature of the Siemens-Martin steel, when welded, — 10°, + 25° and 50° C.



Festigkeitsverhältnis / Résistances / Strength relationships	α_2	Stumpf / Bout à bout / Butt α_1	Einbrand / Pénétration / Penetration α_1	Kehl / D'angle / Fillet α_1
Ursprung / Origine / Origin	0,85	0,7	0,6	0,35
A:B 1:2	0,95	0,85	0,65	0,45

Fig. 5.

Tensile fatigue strength in relation to the lower limit of stress.



Not pre-heated, formation of Martensite.

Pre-heated, no formation of Martensite.

Fig. 6.

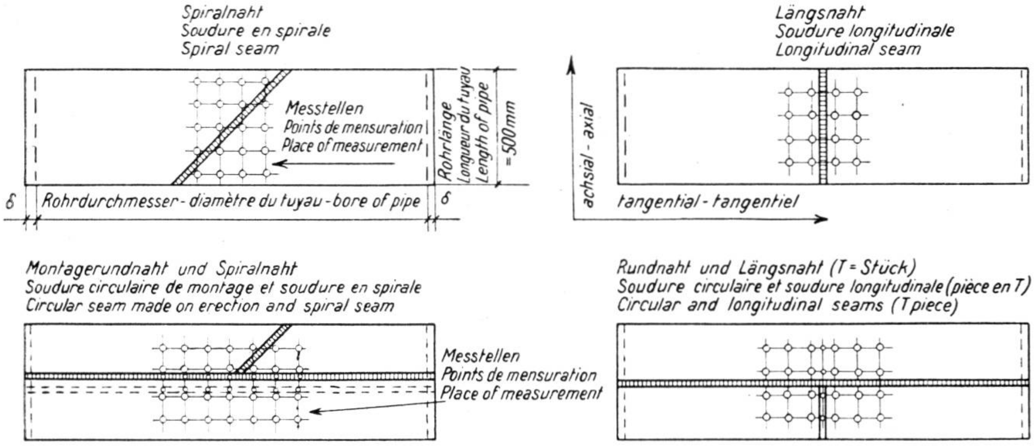
Electrically welded cast steel, carbon content 0.28 %.

Welding without pre-heating:

High hardness number in the transition zone, formation of Martensite.

Welding with pre-heating:

Normal hardness number, no formation of Martensite.



Nature of weld seams, where measured.

Fig. 7 a.

Internal stresses of weld seams, annealed and not annealed.

Maximum values of internal stresses as measured.

annealed or not	Nature of weld seams where measured	Reduced internal tensile stresses in kg/cm²	
		axial	tangential
annealed	Longitudinal X-seam	+ 1010	+ 1060
not annealed	Longitudinal X-seam	+ 1620	+ 2460
annealed	Spiral X-seam	+ 280 + 447	+ 727 + 336
not annealed	Circular and longitudinal U-seams	+ 2070	+ 2070

Fig. 7 b.

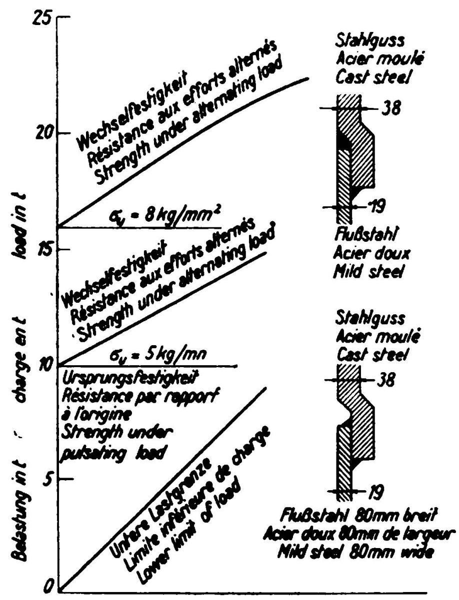
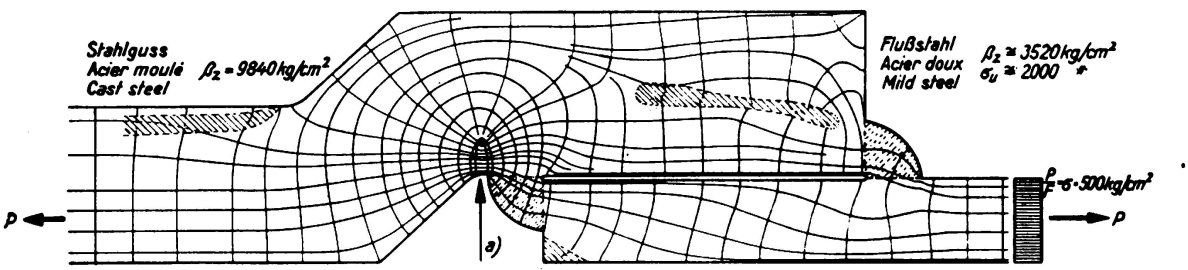


Fig. 8 a.

Increased resistance to alternating stress through suitable weld connections between cast steel and mild steel.



a) Navier-Hooke stresses, by calculation $\frac{P}{F} + \frac{M}{W}$: $\sigma_{max} = 2{,}8\,\sigma = 1400$ kg/cm².

Resistance of the connection to pulsating stress: $\sigma_u \cong 500$ kg/cm².

Concentration of stress at base of notch as determined optically: $\sigma_{max} = 5\,\sigma = 2500$ kg/cm².

Fig. 8 b.

Stress conditions at base of notch.

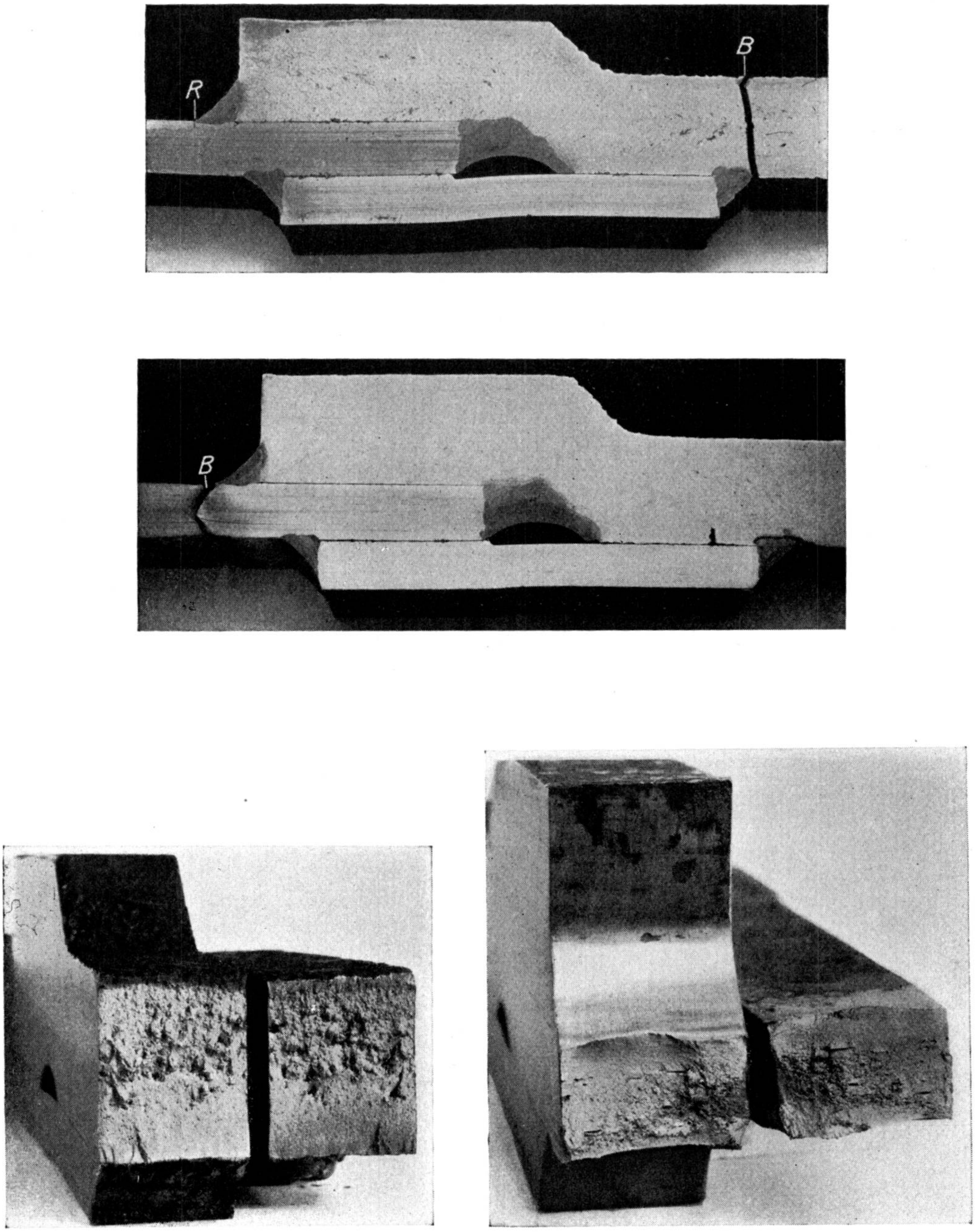


Fig. 9.

Welded connections between cast steel and mild steel.
Increased resistance to pulsating stress by grinding the weld seams.

IIIa 5

The Fatigue Strengths of Welded Connections in St. 37 and St. 52.

Dauerfestigkeit von geschweißten Verbindungen von St. 37 und St. 52.

La résistance à la fatigue des assemblages soudés en acier St. 37 et St. 52.

Ir. W. Gerritsen and Dr. P. Schoenmaker,
i. Willem Smit & Co's Transformatorenfabriek N. V. Nijmegen, Holland.

It has been shown in previous experiments that the fatigue strengths obtained in laboratory experiments with round and flat bars of St. 37 which have been worked smooth, are the same as those of the unwelded material as rolled, but in the case of St. 52 they are lower than the latter. Hence in St. 37 the fractures occur outside and in St. 52 within the weld, but in spite of this the fatigue strength of the welded connection of St. 52 is at least 30 % higher than that in St. 37. Since, moreover, the permissible stresses in St. 52 are 30 % higher than those in St. 37, the same should be true of the welded connections, provided that heavy coated, high quality electrodes have been used.

The values determined are shown in Table 1.

Table 1.
Bending fatigue strengths for welded and unwelded St. 37 and St. 52.

Material	Fatigue strength kg/mm²	Breakage
(a) Bending fatigue tests on round specimens.		
St. 37 — unwelded	$\sigma_{wb} = 20.1$	—
— welded	$= 20.1$	Outside
St. 52 — unwelded	$\sigma_{wb} = 30.8$	—
— welded	$= 26.4$	Inside
Deposited weld metal	$\sigma_{wb} = 24.3$	—
(b) Bending fatigue tests on flat specimens.		
St. 37 — unwelded	$\sigma_{wb} = 17.8$	—
— welded	$= 17.8$	Partly in weld
St. 52 — unwelded	$\sigma_{wb} = 30.5$	—
— welded	$= 22.5$	In weld
(c) Torsion fatigue test on round specimens.		
St. 37 — unwelded	$\sigma_{w} = 11.5$	—
— welded	$= 11.5$	Outside
St. 52 — unwelded	$\sigma_{w} = 17.2$	—
— welded	$= 15.5$	Insidc
Deposited weld metal	$\sigma_{w} = 15.3$	—

These results, while interesting for purposes of comparison, are of little practical importance, for the conditions in practice are quite different, the majority of connections in welded bridges and building structures not being worked over, with the result that non-uniform distribution of stress occurs and these effects are further increased by notch action at the edges of the weld or at the base of fillet seams. There are, therefore, two factors which play a decisive part, namely 1) the execution and workmanship of the weld. 2) The design of the connection.

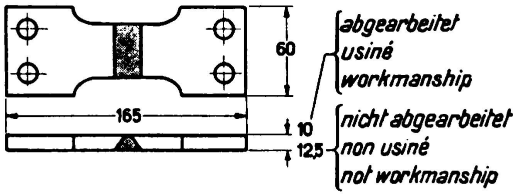


Fig. 1.

Shape and dimensions of flat bending specimen for fatigue tests.

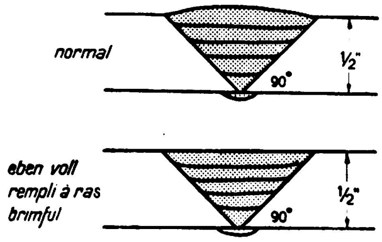


Fig. 2.

Execution of welds.

The effect attributable, to the execution and workmanship of the weld was examined by the authors with the aid of flat bending test bars as indicated in Fig. 1, the weld itself being carried out either in the usual way or smoothed (see Fig. 2), while in a third experiment the upper surface of the bead was filed off. The results of these experiments are shown in Table 2 together with values for unwelded material of three different types of St. 52 — a chrome-copper steel (I), a manganese-silicon steel (II), and a manganese steel (III).

Table 2.

Fatigue strengths of welded connections in St. 52.

Steel	Not welded. Upper surface filed	Welded		
		Weld filed	Not worked	
			Welded in ordinary way	Filled smooth
I	31.0 kg/mm²	23.0 kg/mm²	11.0 kg/mm²	15.0 kg/mm²
II	29.0 „	21.5 „	9.5 „	16.5 „
III	31.5 „	22.5 „	8.0 „	14.0 „

In the case of all the bars which have not been smoothed fracture occurs at the junction of the weld metal and the plate material, either on the upper side of the V seam (Fig. 3) or on the root side at the edge of the backing bead (Fig. 4); a circumstance which may be explained by changes in the micro-structure caused by the more or less remarkable hardening effect at these places (Fig. 5). The efforts of the steel maker will, therefore, be directed towards limiting this increase in hardness as much as possible, but as the phenomenon is connected with the increased strength of these steels it cannot be entirely avoided. The most favourable results were found in the chrome-copper, chrome-molybdenum and manganese-silicon steels when the amount of alloy element present

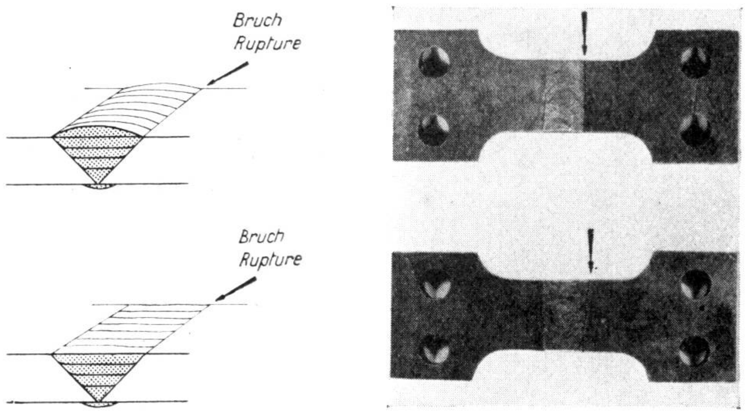


Fig. 3.

Fatigue failures at transition from weld to plate.

was as low as possible, and especially when the carbon content did not exceed 0.15 to 0.20 %.[1]

The effect of shape was examined in a T-connection carried out in several different ways and tested under dynamic loading simultaneously with static pre-

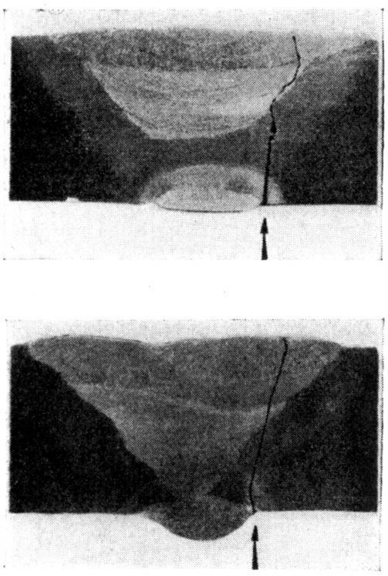


Fig. 4.

Beginning of fatigue failure at edge of reverse bead.

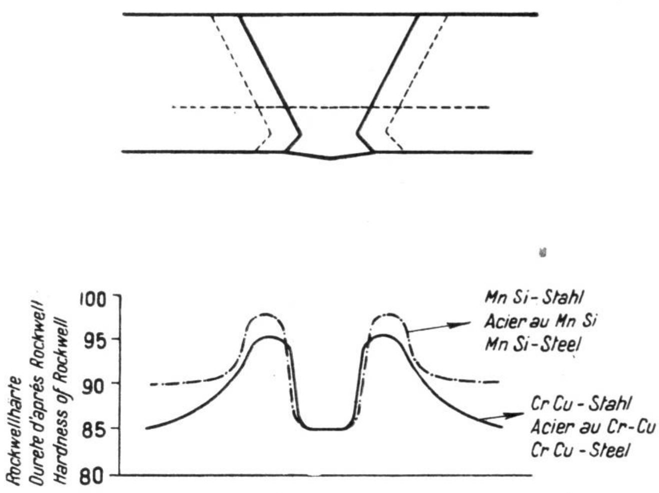


Fig. 5.

Distribution of hardness in a welded connection in St. 52.

stressing. In accordance with the method of calculation adopted by the bridge construction bureau of the Netherlands Railways the dynamic stress was taken as

[1] See Smit-Laschtydschrift, Vol. 1, N° 2 (1937).

30 % of the static pre-stress, and it was sought to obtain in each connection the maximum value of this pre-stress which did not result in fracture after two million changes of load (see Fig. 6).

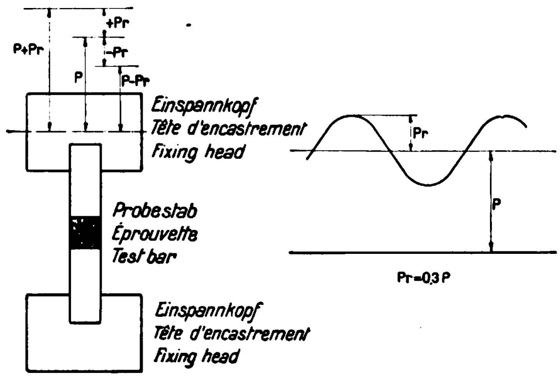


Fig. 6.

Variations of stress in experiments on stress alternating between tension and compression.

The experiments were carried out in a Losenhausen pulsating machine. The T-connection was made in the form of a double-sided fillet weld and as an X-weld (Fig. 7 and 8). The results of the experiments are given in Table 3.

Table 3.

Results of fatigue tests on welded connections in St. 37 and St. 52.

Material	Connection	Maximum statical pre-stress kg/cm²	Dynamic stress	Stress changes without fractures
St. 37	X-weld (Fig. 7a)	1900	±30 %	$2 \cdot 10^6$
	Fillet weld (Fig. 7b)	1250	±30 %	$2 \cdot 10^6$
St. 52	X-weld (Fig. 7a)	2000	±30 %	$2 \cdot 10^6$
	Fillet weld (Fig. 7b)	1000	±30 %	$2 \cdot 10^6$

These results indicate that the fillet seams of the connections in St. 37 possess a higher dynamic strength than those in St. 52, but in the case of the X-seams the values are approximately equal. This clearly shows that the fatigue strength of welded connections in St. 52 is not greater than in St. 37, a fact which may be explained by the greater notch sensitiveness of St. 52. Moreover in both cases the strength of X welds is much greater than that of the fillet seams, and the general rule holds good, therefore, to *adopt butt welded connections wherever possible.*

When fillet welds cannot be avoided they are best made as shown in Fig. 9, the weld having the maximum possible depth, and being made smooth at the edges with a gradual transition into the parent metal without any notches.

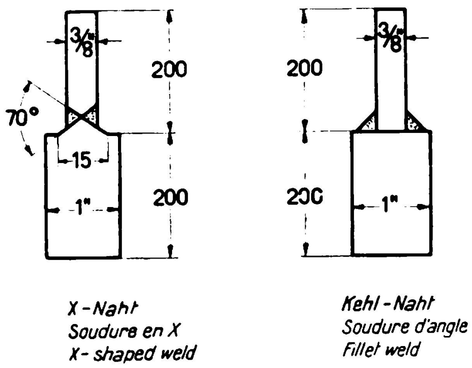


Fig. 7.

Shape and dimensions of specimens for tensile-compressive alternating stress experiments.

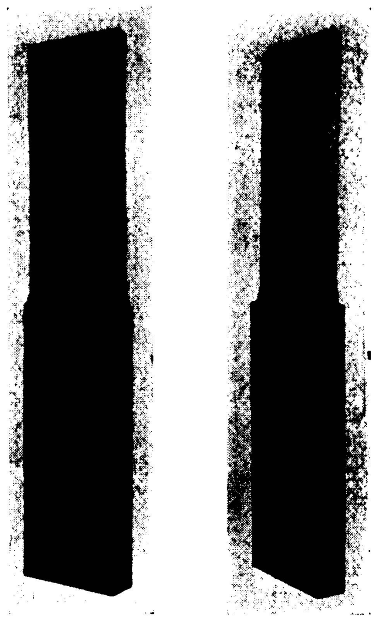


Fig. 8.

Welded specimens for experiments as in Fig. 7.

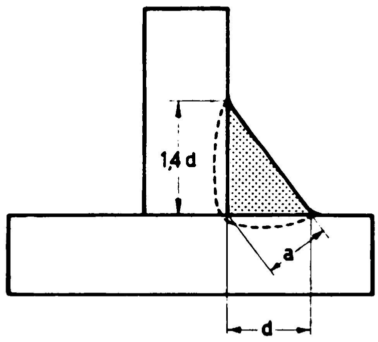


Fig. 9.

Ideal form of fillet seam offering maximum resistance to static and dynamic loading.

The author's experiments were further extended to fatigue tensile tests with shear stresses, but as these have not yet been completed the results will be reported later.

IIIa 6

Notes on the Method of Expression of Allowable Unit Stress as determined by the Pulsation or Reversal of the Stresses.

Angaben über die Methode des Ausdrucks einer zulässigen Spannung, die durch Schwingung oder Wechsel im Vorzeichen der Spannungen bestimmt ist.

Notes sur la méthode d'expression de la contrainte admissible déterminée à partir de la pulsation ou de l'alternance des efforts appliqués.

Jonathan Jones,
Chief Engineer, Bethlehem Steel Co., U.S.A.

In standard American specifications for riveted bridges, a reduction of allowable unit stress has been prescribed in the case of reversed stresses, but not in the case of pulsating stresses (fluctuating but without change of sign).

However, the committee assembled by the American Welding Society to prepare a specification for welded bridges (issued 1936), decided from the available data, largely the published reports of Professors *Graf* and *Schaechterle,* to make a reduction in the allowable unit stress on certain types of welded joints for stresses pulsating through considerable range, as well as for those actually reversing.

These notes do not discuss the actual values selected for allowable unit stress under various conditions, but only the manner of their expression. As most of the important members of any bridge, and their connections, are subject to pulsating stress, it is important to keep to a minimum the arithmetical labor involved in carrying out any prescribed rules.

The previous American specifications applicable, as noted above, in the case of reversed stresses only, require the calculation from the minimum and maximum total stresses of a third or hypothetical stress, greater than the maximum, to which the normal unit stress is applied, thus giving an increase of required area. An identical method is, for geometrical reasons, not possible of employment when reduction of unit stress is to be made for pulsation as well as for reversal.

The official German method ("gamma method") is similar to the foregoing, and requires the calculation from the maximum and minimum total stresses of a multiplier "gamma" to be applied to the maximum stress.

Each of these methods introduces an auxiliary step, the calculation of a modified or hypothetical maximum stress, before proceeding to the determination of the required area. The method adopted by the American Welding Society eliminates the preliminary step and yields the required area by the direct application of a simple formula, the derivation of which will now be given.

Let the line A B C be a graph of permissible unit stresses, plotted so that any minimum is an abscissa and the corresponding maximum is the corresponding ordinate. Thus at A, min = max, and the ordinate S_{-1} is the unit stress selected on the basis of the test data to be permitted in the case of full reversal. So at B, min = 0, and the ordinate S_0 is the unit stress selected to be permitted in the case of pulsation from zero.

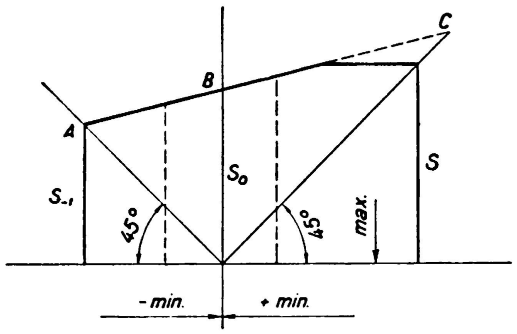


Fig. 1.

For all practical purposes, A B C may be taken as a straight line. It is undesirable to complicate design requirements by introducing any other form of variation, considering how small the percentages of error, even if they could be definitely known, must be.

The full range of the line B C is not utilizable, because the values of maximum unit stress increase beyond the value S which has been established for static conditions (max = min). The sloping line must therefore be neglected above its intersection with a horizontal through the value S.

Consider now a bridge part which undergoes some other degree of reversal than complete reversal, or some other degree of pulsation than return to zero, as indicated by dotted ordinates in the sketch. For this member the unknown "max" and "min" permissible unit stresses are of course proportional to the known "Max" and "Min" or calculated total stresses.

Then:

$$\text{max.} = S_0 + \frac{S_0 - S_{-1}}{S_{-1}} \text{ min.} \tag{1}$$

$$= S_0 + \varnothing \cdot \text{min.},$$

"$\varnothing$" being the slope of the line A B $= \dfrac{S_0 - S_{-1}}{S_{-1}}$.

$$\text{Required area A} = \frac{\text{Max.}}{\text{max.}} = \frac{\text{Max.}}{S_0 + \varnothing \cdot \text{min.}} = \frac{\text{Max.}}{S_0 + \varnothing \cdot \dfrac{\text{Min.}}{\text{A}}}$$

$$\text{A} \cdot S_0 + \varnothing \cdot \text{Min.} = \text{Max.}$$

$$\text{A} = \frac{\text{Max.} - \varnothing \cdot \text{Min.}}{S_0} \tag{2}$$

This, then, is the form in which the specification is cast. For each particular type of stress or type of joint, the committee has selected a permissible value of S_0 and a value of S_{-1}. From these, by Eq. (1), $\varnothing$ is derived and then Equation (2) is written into the specification. The designer, having from the prescribed loding calculated "Max" and "Min" derives his area "A" in the simplest possible fashion.

In the future, as further test results become available, and as for other reasons the necessary factors of safety are re-considered, future committees may modify "S_0" or "S_{-1}", or both. The form of the several formulas need not be disturbed, and a simple modification of "$\varnothing$" or "S_0", or both, will embody the desired change or changes.

As an example the American Welding Society specification for the area of fillet welds is:

$$\text{Area} = \frac{\text{Max.} - \frac{1}{2}\,\text{Min.}}{7200} \text{ but not less than } \frac{\text{Max.}}{9600}.$$

(The second expression embodies the portion of the foregoing diagram in which the sloping line is replaced by the horizontal line through the ordinate "S".)

Example 1. Max. $= +\ 80000$ Min. $= -\ 80000$

$$A = \frac{80\,000 + 40\,000}{7200} = 16.7 \text{ sq. in.}$$

Example 2. Max. $= +\ 80000$ Min. $= -\ 40000$

$$A = \frac{80\,000 + 20\,000}{7200} = 13.9 \text{ sq. in.}$$

Example 3. Max. $= +\ 80000$ Min. $= 0$

$$A = \frac{80\,000}{7200} = 11.1 \text{ sq. in.}$$

Example 4. Max. $= +\ 80000$ Min. $= +\ 16000$

$$A = \frac{80\,000 - 8000}{7200} = 10.0 \text{ sq. in.}$$

$$\text{but not less than } \frac{80\,000}{9600} = 8.33 \text{ sq. in.}$$

Example 5. Max. $= +\ 80000$ Min. $= +\ 64000$

$$A = \frac{80\,000 - 32\,000}{7200} = 6.67 \text{ sq. in.}$$

$$\text{but not less than } \frac{80\,000}{9600} = 8.33 \text{ sq. in.}$$

III b

Design and execution of welds with special consideration of thermal stresses.

Berücksichtigung der Wärmespannungen bei der baulichen Durchbildung und Herstellung geschweißter Konstruktionen.

Disposition et exécution des constructions soudées en tenant spécialement compte des contraintes dues aux variations de la température.

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IIIb 1

The Influence of Metallurgical Factors on the Safety of Welded Structures. (Stresses and Cracking Tendency.)

Zur Frage des metallurgischen Einflusses auf die Sicherheit geschweißter Bauwerke. (Spannungen und Rißneigung.)

L'influence de la composition métallurgique sur la sécurité des ouvrages soudés. (Contraintes et tendance à la fissuration.)

Prof. Dr.-Ing. E. H. Schulz,
Direktor des Forschungsinstitutes der Vereinigte Stahlwerke A.-G., Dortmund.

It may be laid down as a first principle that the reliability of welded structures depends mainly on the design and execution of the welds. In addition, large demands are made on the metallurgist as regards the mechanical properties of the structural steels supplied by him, especially those steels which are heavily stressed, and strict conditions are imposed as to the composition of the latter. Even if the influence of the steel maker on the safety of the structure may appear limited, there is, nevertheless, evidence of a relationship between the chemical composition of the steel and the development of stresses, as well as the tendency to cracking in a welded structure. Here it should be emphasised that such evidence, to be of practical use, can only with difficulty be obtained through laboratory experiments, as the conditions are too complicated. Much has been achieved, however, through practical observation and experience — that is to say, methods which are of real practical use and which throw light on the nature of the steel alloys to be employed — and this knowledge has been won through an intensive and sustained collaboration between metallurgists, welding technicians and designers, continuously and exhaustively applying the results of modern research in all relevant fields.

There is an important consideration which arises in reference to this question, a principle which plays an important part in almost every branch of the application of steel, but which is too often insufficiently observed. The principle is as follows: the higher the grade of the steel — in other words, the greater its utility, and the better it is made — the greater the care with which it must, as a rule, be treated in the later stages. An example is ready to hand: St. 37, which is as simple a material as could be as regards composition and method of manufacture, admits of being worked and heat-treated, etc. within much wider limits than was permissible for St. 48 (a material which has since disappeared from use), even though the latter represented no very great advance over St. 37 in the development of structural steels from the point of view of their stress capacity. From the metallurgical point of view the step to St. 52 was a much greater one, for whereas St. 48 was still purely a carbon steel, St. 52 is alloyed. For this very reason St. 52 must be regarded as a more sensitive material in welding. It follows that in the construction of welded structures using St. 52 the rules of correct welding design and of careful welding workmanship must be even more strictly adhered to than with St. 37.

As regards the metallurgy of St. 52, it is further to be noticed that different compositions and different methods of alloying are practiced in different steelworks, but when comparative experiments are made and differences are discovered in the performance of two kinds of St. 52 these cannot always be attributed to differences in composition or alloying. There can be no doubt that apart from differences in these respects an important part is played by differences in the melting and working procedure which obtains in the different steelworks as a consequence of differences in their equipment and general methods.. These are influences which cannot all be identified in the finished steel.

The experience available in the author's institution regarding the connection between the performance of welded structures and the metallurgy of the steel employed has reference mainly to two kinds of St. 52; one alloyed for the most part only with silicon, the other also with chromium and copper. In the last mentioned, which is the Union-Baustahl, the silicon and manganese contents also, of course, play a part. The experience available in respect of the chrome copper steel is much richer than that with the silicon steel, as the preparation of the latter was very soon found to be exposed to grave defects. For this reason the author's information is based mainly on experience with St. 52 alloyed with copper and chromium.

Moreover, it is perhaps a matter of some importance that this experience has not been derived from a single source concerned only with some particular aspect, but constitutes a summary of experience obtained in the steelworks, the welding workshop and the bridge fabricating shop in collaboration with the Research Institute of Kohle- und Eisenforschung (Vereinigte Stahlwerke).

There are two ways in which the chemical composition of St. 52 would appear to be important as affecting the stress conditions and susceptibility to cracking in welded structures.

It is known that welding gives rise to stresses in the structure which may, in certain circumstances, result in additional stresses difficult to control, and may lead to fractures. The question arises how far the composition of the steel may influence the magnitude of these welding stresses: for it is not reasonable to suppose that notable stresses arise with one kind of steel under certain conditions and none at all with another kind, and the question can, therefore, only be one of difference in the orders of magnitude of the stresses.

An important criterion for the development of these stresses would appear to be the hot yield point of the steel, and this depends in turn on the nature and quantity of the added alloying components. Yet, in another field of work, that of the construction of heavily stressed boilers, it has been found possible to increase the high temperature strength by suitable alloying, to a quite remarkable extent compared with the strength of the ordinary carbon steel. It might have been supposed that as a rule the stresses arising in the welding operation would increase as the hot yield point was increased. Yet careful stress measurements, carried out on the chrome copper steel used by the author, disclosed practically no higher stresses, even under a great variety of experimental conditions, than were found in St. 37, despite the greater high-temperature strength of the former.[1] In one series of experiments, indeed, the stresses in St. 52 were found to be less than those in St. 37: but this result was not found to be generally valid. In no case, however, was a stress found to exist which might be regarded as dangerous by comparison with that in St. 37.

Conditions from the second point of view are somewhat more complicated.

In welding, the zones close to the weld seam are heated to a very high temperature, and if the thickness is considerable the large amount of heat produced in these zones is very rapidly conducted away by the neighbouring cold zones. In other words, an effect may be produced in the highly heated zones which corresponds, or at any rate approximates, to sudden quenching. In this way "hard spots" may be produced in the welded construction, and these hard spots are brittle. It is true they may have a high tensile strength, but under bending stress they tend to crack. Indeed, the hard spots may by themselves lead to cracking, on account of the volume changes which attend the transformations even without the presence of any external load.

The hardening capacity of St. 52 would appear to depend in the first place on its *carbon content.* In accordance with the relevant regulations of the German Reichsbahn the carbon content in St. 52 is limited to 0.20 % for thicknesses up to 18 mm and to a maximum of 0.25 % for still greater thicknesses, this being done in direct reference to weldability. There might be other inducements to the metallurgist to increase the carbon content, especially since strict compliance with the limits imposed thereon makes it difficult to obtain the prescribed yield point (minimum 36 or 35 kg/mm^2). All our experience, however, is to the effect that with due regard to the safety of welded constructions the limit 0.20 % of carbon should not be exceeded. As early as 1933 *Buchholtz* and the author showed that the hardening capacity of the parent material, and especially its carbon content, has much influence on the fatigue resistance of welded St. 52.[2] Thus welded connections made with St. 52 containing 0.24 % of carbon were 20 to 30 % less strong than those made of St. 52 showing the same mechanical strength but containing 0.16 % of carbon. We established the fact that it is desirable, where possible, that the carbon content should not be allowed to exceed 0.18 %. It should be clear, however, that a limit on the carbon content — and still more, as will be explained below, on the other alloy constituents — is to some extent bound to be detrimental to the yield point.

The part played by the other alloy constituents in St. 52, from the point of view of weldability, is more difficult to ascertain, and the conditions in regard to *silicon* appear particularly complex. Silicon in itself does not tend to any great extent towards an increase in hardening capacity, but the dislike of most steelworks for a pure or only slightly altered silicon steel is due to other reasons.

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[1] *H. Bühler* and *W. Lohmann:* Elektroschweißung 5 (1934), p. 226.

[2] Stahl und Eisen 53 (1933), p. 545/52.

In spite of this, the silicon content has for some time been limited to a maximum of 0,4 %, and the author would wish particularly to emphasise that this is so despite the preparation of a large number of meltings with a higher silicon content, approximately 0.6 %. It was quite obvious, however, that the steelworks experienced difficulties with this high silicon steel, with the result that the limit of 0,4 % is now strictly adhered to.

The maximum content of *manganese* which we allow is 1.1 %. According to the experience of some other works 1.5 % is permissible, but doubtless only with a correspondingly smaller content of other alloy constituents. In this connection it may be mentioned that according to *Sandelowski*,[3] and also according to *Schulz* and *Püngel*,[4] electrodes with a high manganese content tend to a greater amount of shrinkage in the seam and greater stress in welding.

Copper is added to the Union-Baustahl up to 0.8 %, without any difficulties or disadvantages from the point of view of the safety of welded connections being disclosed. In this connection it is of interest to note a recent publication by *S. Epstein, I. H. Nead* and *I. W. Hally*,[5] who, in attempting to develop a weldable steel with a good strength at high temperatures, arrived at the following composition:

	C	Si	Mn	Cu	Ni	P
Maximum:	0.10	0.15	0.50	1.00	0.5	0.12

Here the limit of carbon content is very much reduced. The manganese and the silicon are also kept low, while a copper content of 1 % is regarded as advantageous. The high content of phosphorus is also of interest from this point of view.

The presence of *chromium*, when it exceeds certain limits, gives rise to undesirable effects in welding, and chromium also tends to hardening. Regarding this matter of the influence of the chromium content we are in possession of very extensive observations, and from these the rule has been laid down that the chromium content should not be allowed to exceed 0.4 %. Up to this limit, the presence of chromium has not given rise to any difficulties. In other works a higher limit is allowed, and mention may be made particularly of the British Chromador steel, wherein, to the author's knowledge, the chromium content is as much as 0.8 %. It would be of interest to learn what experience has been obtained in the welding of this steel.

Molybdenum, within the limits that it is present in St. 52, may doubtless be ignored from the point of view of the quality of welds.

In reproducing the chemical composition proposed by *Epstein* and his collaborators special attention was drawn to the phosphorus content, which reaches the high value, according to our ideas, of 0.12 %. We now know, on the basis of work carried out in our own company, that very often the phosphorus content of steel is not attended by the grave disadvantages which are frequently ascribed

[3] Elektroschweißung 2 (1931), p. 48/53.

[4] Stahl und Eisen 53 (1933), p. 1233/36.

[5] American Institute of Mining & Metallurgical Engineers, 1936, Technical Publication No. 697; Metals Technology, 1936, Vol. 3, April.

to it, but that on the contrary, under certain conditions, it may be regarded as a useful and desirable element. For our steel St. 52, the author would not indeed regard so high a phosphorous content as in the American steel as desirable[6]: but to say this is not to imply anything against the American steel, for it would appear that the danger from phosphorus decreases in proportion to the carbon content, and in the American steel the latter is very low. It would be of great interest to learn more about the behaviour of this American steel in welding.

In the German St. 52 the phosphorus content is manifestly so low, since it is a basic Siemens-Martin steel, that as regards the performance in welding and on the site no difficulties can arise through its presence. The same is true in regard to the sulphur content.

Addendum.

While the report was in the press, regulations were issued by the German Reichsbahn allowing increased amounts of alloy in St. 52, the main object of this being to obtain a better guarantee of weldability while at the same time approximating the various kinds of St. 52 more closely to one another as regards composition. In accordance with these regulations the following upper limits are laid down for the alloy constituents:

Carbon	maximum 0.20 %
Manganese	maximum 1.20 %
Silicon	maximum 0.50 %
Copper	maximum 0.55 %.

In addition to these constituents the St. 52 may receive a further addition of either chromium up to a maximum of 0.40 % or of molybdenum up to 0.20 %. Finally, an additional manganese content of 0.30 % may be present, bringing the total content of manganese up to 1.50 %, but only in the absence of either chromium and molybdenum.

At the same time, however, the prescribed minimum yield points in respect of the greater thicknesses were stepped down.

[6] *K. Daeves, A. Ristow* and *E. H. Schulz:* Stahl und Eisen 56 (1936), p. 889/99 and 921/27.

IIIb 2

Stress and Distortion Due to Welding.

Schweißspannungen und Verwerfungen.

Contraintes internes et distorsions provoquées par la soudure.

W. Heigh,
Welding Superintendent, Babcock & Wilcox, LTD., Glasgow.

One is apt to glean from the leading papers in this discussion that welding in its present state is a very inexact science. There are too many doubts and fears — too many unknown factors.

Those of us who are concerned chiefly with getting work done cheaply, quickly and well, detest the unknown and avoid it at all costs.

Some reassurance seems to be necessary.

For example, the fear is expressed in one of the papers that the weld metal may be in a dangerously chilled condition and that it is liable to be dangerously stressed.

Since the weld metal obtained from certain good and quite cheap covered electrodes is consistently ductile, no real danger exists. An elongation of 22 to 25 % on a gauge length of four diameters is commonly obtained in the as-welded condition.

Dangerously chilled metal could not be so ductile, while with so much stretch left in the weld metal the stressed condition is much less important than it seems.

We are also warned to be careful of the stressed condition in the parent material — the metal in the pieces being joined together.

But rolled steel sections and mild steel plates are almost invariably cold straightened before being applied to important structures and are therefore in a stressed condition. Hundreds of very high self-supporting steel chimneys (that is, chimneys without guy-rope supports) sway in high winds over populous workshops and cities; yet no one thinks of excessive danger to life although all the plates have been cold rolled and therefore stretched far more than any weld-stretched piece of metal.

If inequality of stress distribution is feared, the analogy of hot bent rolled sections is a comfort. Those are used freely in important structures, yet the fact that most of the bends are made by local heating and forming leaves the metal in a condition precisely similar to that in a weld and the surrounding material.

Speaking practically, all those fears may easily be exaggerated. If a good electrode and a sound welding procedure are chosen and if the chilling effect of large masses of material, very cold weather and weather-exposed welding positions are allowed for by slight preheating welding may be used with full confidence.

It is, of course, an advantage to relieve the stresses by heat treatment and in certain cases, notably in thick welded pressure vessels, stress relief is a normal part of the manufacturing procedure. Apart from the extremely high pressures frequently involved (a sufficient argument for special treatment) the stressed condition of the cold formed thick plates and the locked condition of many of the welds in pressure vessels are such as are seldom if ever met with in structural work.

From another point of view, the desire is general in design offices and workshops for quick safe rules for the control of distortion in the final shape of a structure. Some suggestion are offered by the leaders in this discussion. Possibly those which follow will add something of value. Distortion in those notes refers

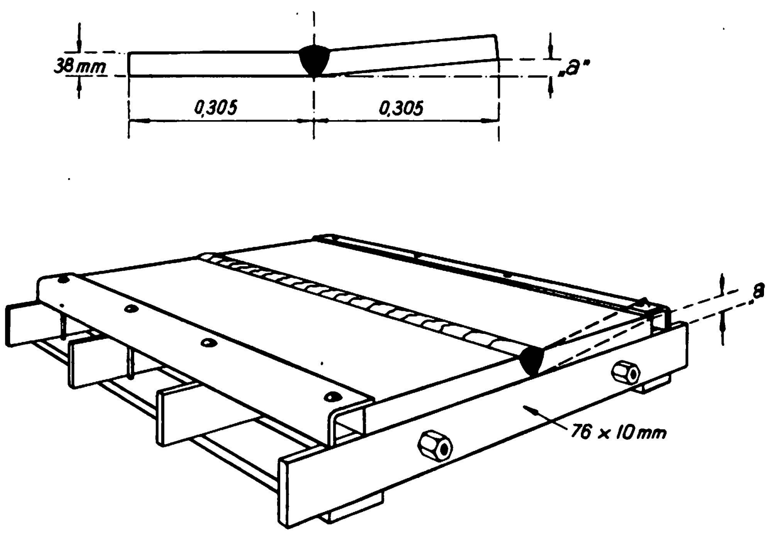

Fig. 1.

to the visible change of shape, not to the displacement within the material which is its root cause.

1) Clamping need not be rigid to be effective. For example, Figure 1 shows two test plates in position in a clamp designed to reduce the distorsion. Welding the plates free, dimension "a" was 1″ (25 mm). Welding the plates in the clamp, dimension "a" was reduced to $^3/_{16}$″ (5 mm).

The moment of resistance of the clamp was one quarter that of a weld of plate thickness, or equal to the resistance of a plate one half the thickness being welded.

No resistance (except friction) was offered to the transverse contraction. This is deemed important.

The flats of mild steel were of such a length that the angular movement due to the shrinkage of each run of welded did not strain the flats beyond the elastic limit.

The principle of the clamp has many applications and has been used freely with success.

2) Size of electrode and method of deposit used are of importance.

Figure 2 illustrates three conditions in a butt weld.

a) N° 8 w. g. electrodes (4 mm diameter) bead runs, gave an angular distortion of 8°.

b) $^1/_4''$ (6,3 mm) diameter electrodes, bead runs, gave an angular distortion of 4°.

c) $^1/_4''$ (6,3 mm) diameter electrodes, woven layers, gave an angular distortion of 3°.

A further test in a clamp gave an average angular distortion of only 1° 11'.

Tests a) and b) were repeated with fillet welds with somewhat similar results.

Large electrodes certainly reduce this very troublesome type of distortion. It is also important to add that welds a), b) and c) were all submitted to mechanical tests and met the requirements of the American Fusion Welding Code for Pressure Vessels so that in this sense the welds were of equal value.

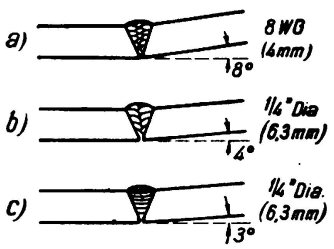

Fig. 2.

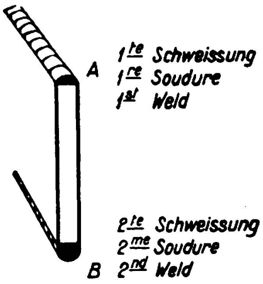

Fig. 3.

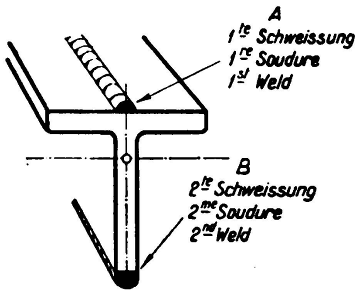

Fig. 4.

3) Rotation of welding assists in reducing distortion.

a) Approximately equal effects are obtained by running two welds, placed symmetrically at an equal distance from the neutral axix of a symmetrical section, simultaneously.

b) If a first run of welding distorts a symmetrical section, a first run of double the volume on the opposite side or edge will approximately balance the first distortion. Figure 3 illustrates a simple test to check this. In multiple run welds distortion from later runs requires a much greater opposing effect.

c) The first in order of two welds in an asymmetrical section as illustrated on Figure 4 should be that nearest the neutral axis.

With training, assemblers and welders acquire considerable skill in checking and controlling distortion. For important non-recurring work, however, the assembly should be planned beforehand and a check kept as the work proceeds so that rotation of welding may be changed, as is found necessary, to correct distortion and keep the assembly in shape.

IIIb 3

Internal Stresses in Welded Joints.

Innere Spannungen in geschweißten Stößen.

Efforts internes dans les joints soudés.

J. Orr,

B. Sc., Ph. D., Glasgow University.

The lecturers have given very thoroughly the disturbing effects of welding in producing distortion and internal stress. They have also spoken on the danger of cracking and of the need for further investigation on the actual weakening effect of the disturbances due to heat and of the internal stresses. Our experience is that the danger of cracking is the cause for real anxiety.

Internal Stress. The writer carried out a series of tests on mild steel and on steels of higher tensile strength (37—43 tons/in.^{2}), the increase being obtained by small additions of carbon, manganese and chromium. They were interesting in that a comparison was made of the welded specimen in the unannealed state with the specimen annealed by heating for a few hours at 600^0 C. In the latter case the internal stresses are removed so that the tests detected any effect due to internal stress.

The tensile strength and the impact value of the joint were reduced a little by annealing; the fatigue strength obtained in a machine capable of testing the complete joint, remained the same; the bend test for ductility in the butt joints showed an improvement in the annealed specimens, but several of the electrodes used gave welds satisfying the standard bend test in the unannealed state. The conclusion from these tests is that internal stresses adjacent to the weld are not a weakness, practically speaking, if good electrodes are used.

Tests on the value of residual stress.

A series of tests was carried out by the writer to find the actual value of internal stress in a severely constrained condition. The arrangement is shown in Fig. 1. Two $^1/_2''$ plates prepared for a butt weld, were first welded to a 3″ thick plate at their ends. They were then welded together. After mounting a tensometer, the plates were sawn through. The reading on the tensometer gave the release of strain and therefore the amount of residual stress. The results were as follows, as shown in Table 1.

Table 1.

Specimen	Length X ins.	Welding	Residual Stress tons/in^2
1	9	With $^{5}/_{16}$" rods	12.0
2	58	„ $^{5}/_{16}$" „	4.2
3	9	„ $^{1}/_{8}$" „	13.0
4	58	„ $^{1}/_{8}$" „	7.2
5	9	Hammered hot	cracked
6	9	„ „	5.0
7	9	„ cold	4.5

The first point of interest in this table is the effect of the length of plate. Increasing the length of plate reduces the stress and emphasizes the point made by the lecturers that there should be flexibility in the part bordering on the welded seam, in this case it is the flexibility of the long plate.

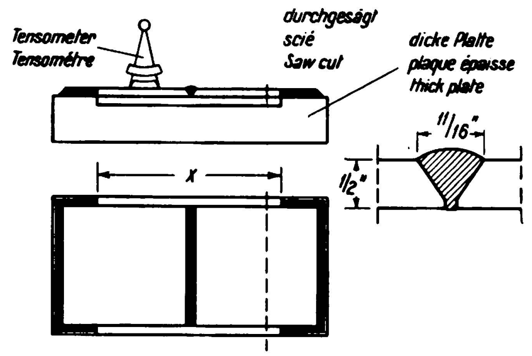

Fig. 1.
Contraction Stress in Butt Welds.
(Plates Welded at Ends Before Butt Welding.)

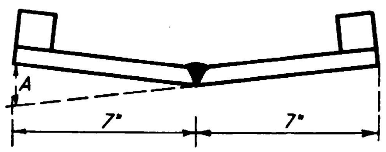

Fig. 2.
Angular Distortion in a Single-V Butt Weld.

The second point is the effect of larger compared with smaller electrodes. These results agree with those quoted by the lecturers, that the larger rods produce less residual stress.

The third point is the effect of hammering, which certainly reduces the stress, but increases the danger of cracking. From later tests using a wide variety of different makes of electrode, the writer is of the opinion (1) that only a few electrodes give weld metal capable of being hammered without the danger of cracking, (2) that the first run of welding should not be hammered as it was shown from hardness tests to be a danger spot and where welds failed, the cracks started from this part, and (3) that the outer layer should not be hammered.

Tests on angular distortion of a single-V butt weld.

These tests are of interest in showing the effect of a small amount of restraint. The restraint was obtained by placing two weights on the plates as shown in Fig. 2. The distortion "A" was measured after cooling and is given in Table 2.

Table 2.

Specimen	Welding	Current amps	Distortion 'A' ins.
1	5 runs $^1/_8''$ rod	110	0.28
2	3 „ $^3/_{16}''$ „	170	0.05
3	3 „ $^3/_{16}''$ „	220	0.044
4	2 „ $^5/_{16}''$ „	340	0.031

Plates $^1/_2'' \times 7'' \times 7''$

The restraining effect of the weights is small as it produces a calculated bending stress in the weld of only $\frac{1}{7}$ tons/in^2. This test confirms the effect of the smaller rods in building up a greater distortion, and therefore where the restraint is more definite, in building up a greater stress.

IIIb 4

Allowance for Temperature Stresses in the Design and Execution of Welded Structures.

Berücksichtigung der Wärmespannungen bei der baulichen Durchbildung und Herstellung geschweißter Konstruktionen.

Les contraintes thermiques dans la disposition constructive et l'exécution des constructions soudées.

Dr. Ing. K. Miesel,
Grünberg.

Professor *Bierett* draws a distinction between those shrinkage stresses which are produced by internal and those produced by external agencies — a distinction which is important not only as regards the distribution of stress in structural members, but also from the point of view of combatting the effects of shrinkage.

The internal stress can be dealt with only by making use of the properties of the weld metal and by control over the welding process, or by mechanical action such as clamping the work and hammering the seams. Annealing, which would be the most effective remedy, cannot be applied in bridge and structural work.

Difficulties from external stress can be met by due attention to the design of the structure, and also at a later stage in the construction. Recently the attention of engineers has mainly been concentrated on the jointing of plate web girders, and Professor *Bierett* shows in his paper how shrinkage may be compensated by the insertion of a previously bent strip of plate in the web.

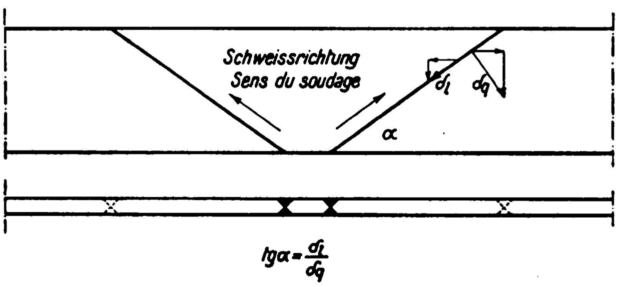

Fig. 1.
Wedge joint for plates.
(Schweissrichtung = welding direction).

In two large structures the flanges were connected by wedge shaped cover straps after the web had been welded (Fig. 1). The angle of bevelling was decided from the relation between longitudinal and transverse shrinkage as found by experiment. As welding was carried out from the narrow to the long side of the wedge, the longitudinal components of the shrinkage were relieved and the transverse components were increased, with the result that the cover plate was drawn uniformly into the joint. This effect was confirmed by preliminary experiments on thin plates.

In the formation of thick flange plates for bridges it was found, however, that the expected action was soon defeated by the internal stresses which arose through the welding of the tulipshaped seams, and when these seams had been filled to

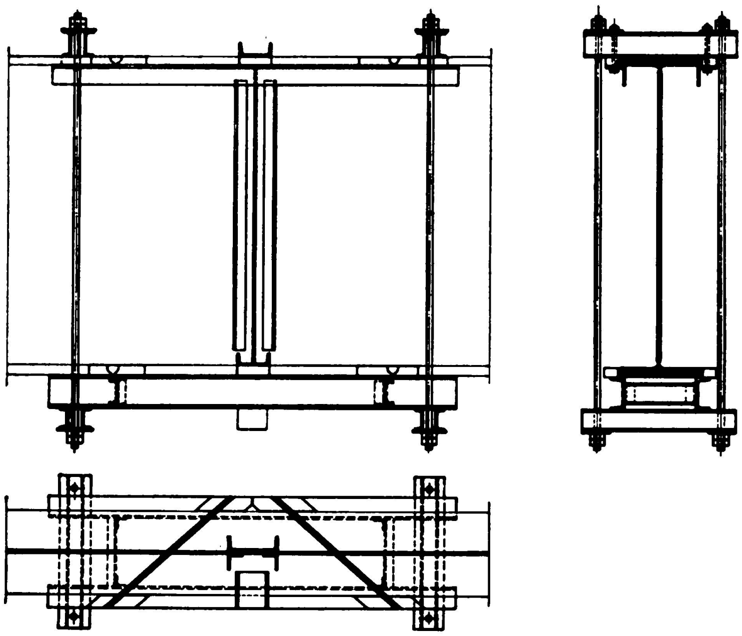

Fig. 2.
Testing arrangement for wedge joint.
Proposal.

two-thirds of their total dephth a powerful angular contraction occurred, which had to be counteracted by continually tightening the clamping on the parts to be joined. Fig. 2 shows the design of a clamping arrangement, and Fig. 3 in-

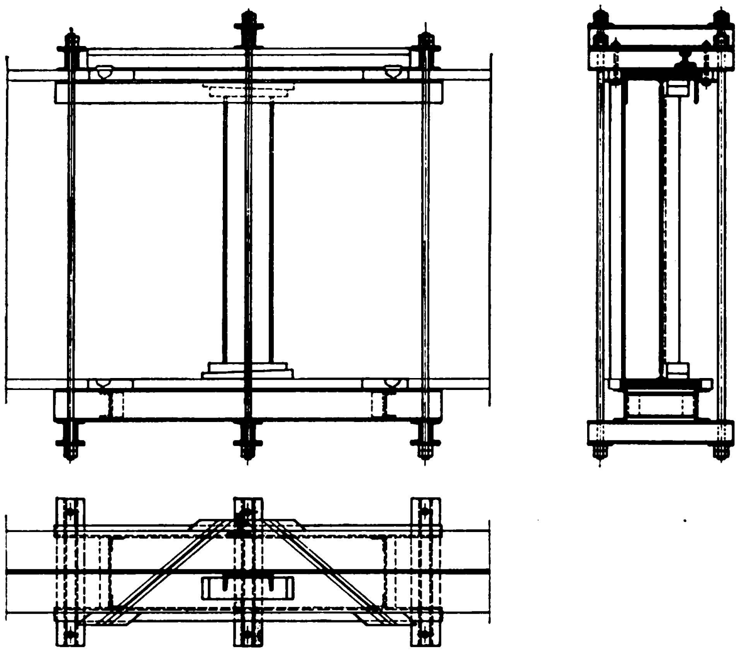

Fig. 3.
Testing arrangement for wedge joint.
Final arrangement.

dicates how strongly this has to be constructed in order to overcome the angular distortion in welding. It was found possible to make girders of St. 37 completely free from shrinkage cracks, but in St. 52 a tendency to these was observable and they were avoided by hammering the second half of the seam, a procedure whereby the direction of the shrinkage can be controlled. It is not yet definitely established, however, whether hammering may exert an unfavourable effect on the mechanical properties of the material.

Shrinkage cracks may also be avoided by adopting a proper sequence of operations in welding, and by pre-heating the parts. In one instance a plate girder of St. 52 was being welded in very hot summer weather when the operation had to be interrupted on account of a hailstorm, and the sudden cooling of the thin web plate, connected as it was to the much thicker flanges, resulted in this being torn away over the whole length of an incompletely welded joint. The latter was re-welded after preliminary heating, and by this means a perfect new weld was obtained.

It may be inferred from a number of publications that the chief part in avoiding shrinkage stresses is played by the designer, but, as these examples show, he is powerless against internal stresses. The external stresses may be reduced by the adoption, where possible, of sufficiently resilient connections. There should be no hesitation in preferring rivetted connections in situations where excessive shrinkage effects are to be apprehended and where rivets are not entirely ruled out by aesthetic considerations. It is a matter in which the demands of the architect may frequently be in conflict with the clear obligation of the engineer not only to secure the most economical arrangement of structural parts, but to combine this with safety and efficiency. Where rivetted connections are so used they may be regarded as playing much the same part as the discontinuities which are introduced into reinforced concrete structures on account of shrinkage effects.

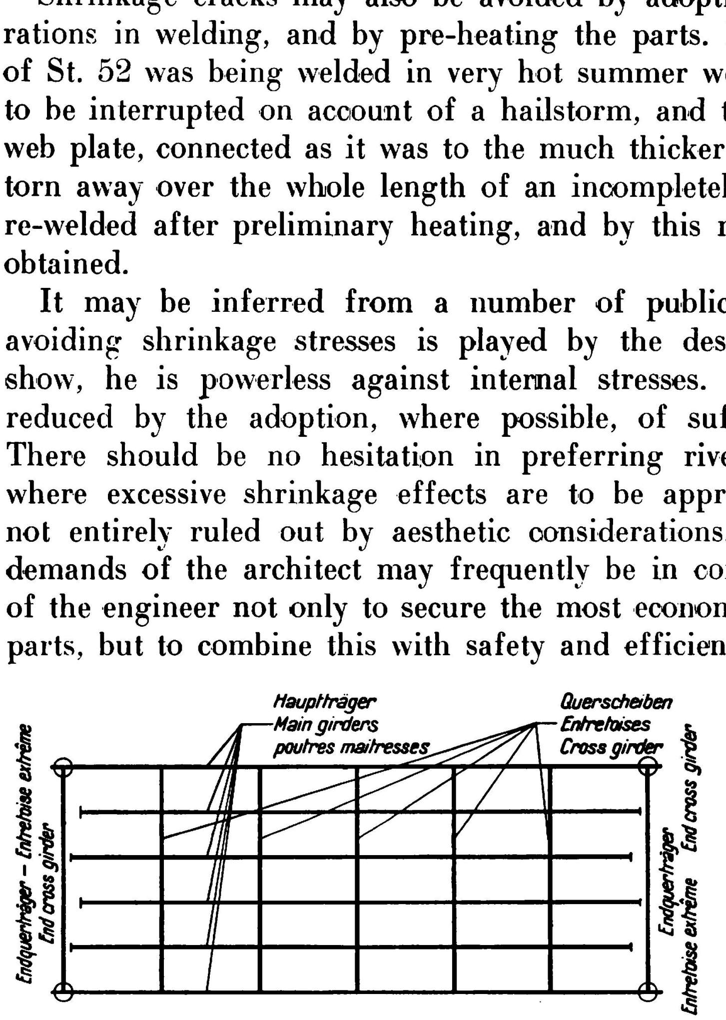

Fig. 4.

Shrinkage of main girders of a grated bridge.

Fig. 4 is a diagram showing a bridge floor in which the main and cross girders were welded on the site. The shrinkages accumulated to different totals on either side of the end cross girders, and this being the case it was found expedient to make the connection between the main girders and the end cross girders by means of rivetting. In this way the cross girders in question are not restricted as to position, and a more accurate alignment of the track is obtained.

Special difficulties attend the construction of members wherein the shrinkage effects are two or three dimensional. Fig. 5 shows the framed main girder of a bridge, and to a larger scale the corner of the frame. The statical stresses at different sections are indicated in the diagram, and the heaviness of the loads which have to be transferred by the fillet welds from the flanges on to the web is made apparent. The excessive thickness which had to be given to these fillet welds was especially conducive to cracking, especially since St. 52 was being used, and such cracking can as a rule only be avoided by hammering.

In this instance, as is true of web plates in general, the shrinkage stresses may lead to bulging, or what is even more dangerous, may be superimposed on other stresses so as to cause failure of the plates. It is to be recommended,

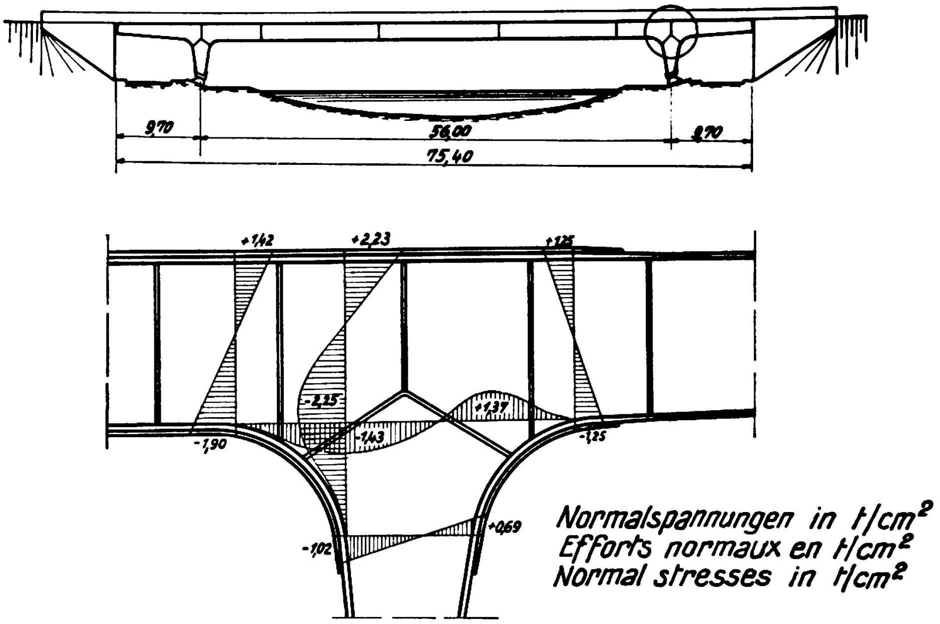

Fig. 5.

Static stresses in frame connection.

therefore, that in the region where the shrinkage stresses occur the stiffeners should be designed to carry the same longitudinal and transverse loads as would occur in a framed girder assumed to take the place of the plate web girder, and should not merely be dimensioned for the degree of stiffness required by the buckling theory.

Observations and measurements carried out in reference to distortion agree in indicating that the shrinkage stresses due both to external and to internal effects may approach the elastic limit. The experimental apparatus shown in Fig. 6 was used for the purpose of measuring shrinkage stresses due to external loading. The test pieces, to be connected by a V seam, were held in place during the welding process by pins fixed into a thick piece of steel, so as to prevent any movement. The free end of the specimen was held in the testing machine and was subjected to tension until it became possible to withdraw the pins by light hammering, thus indicating that the whole of the shrinkage stresses had been

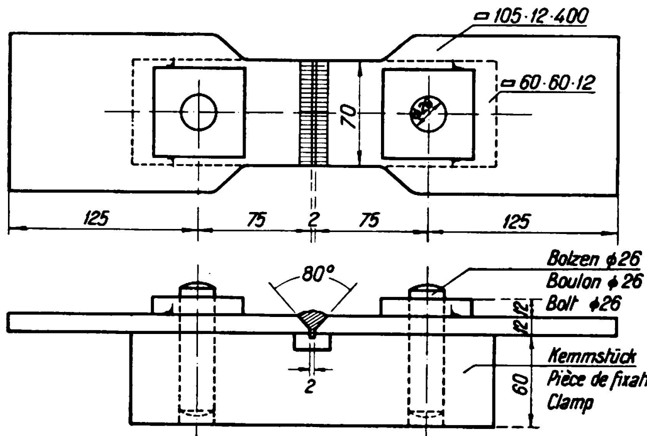

Fig. 6.

Arrangement for measuring shrinkage stresses.

transferred from the pins to the machine. Fig. 7 shows the shape of the stress-strain curves obtained for St. 37 and St. 52, the shrinkage stresses being in the neighbourhood of the limit of elasticity. This also occurs when the specimens are repeatedly loaded and unloaded within the range of the stress that will arise in practice, namely 1.4 to 2.1 tons per sq. mm, before making the experiment. If the specimens are stretched by only a small amount in excess of the shrinkage stress value first measured, the shrinkage stress obtained on a second attempt amounts to only 50 to 75 % of the first value. The values which correspond to this higher degree of tenacity correspond to the upper limit for St. 37 and to the lower limit for St. 52. In this case it could even be observed that the steel

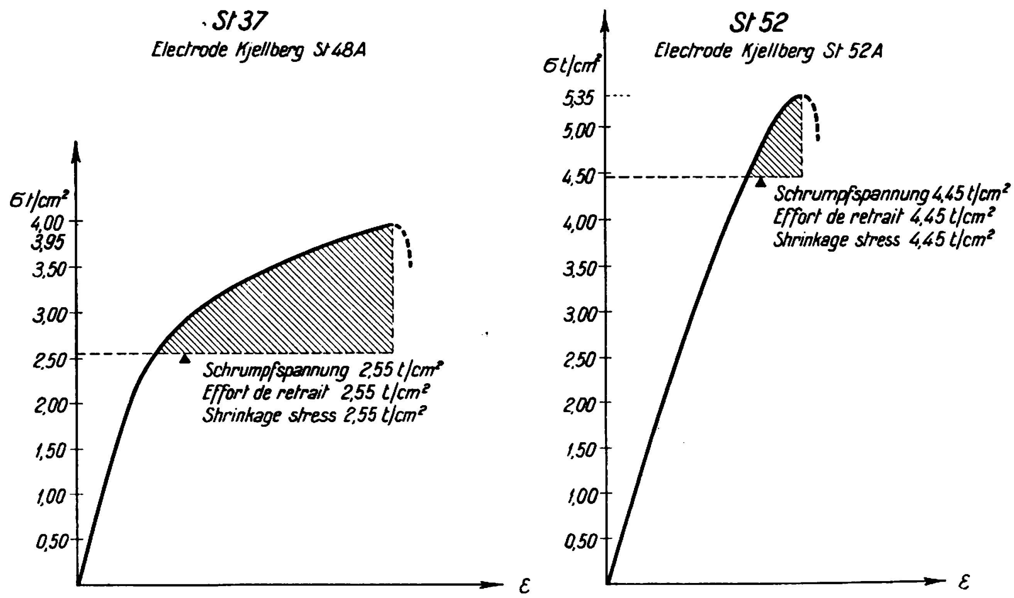

Fig. 7.

Elongation diagrams for shrinkage stresses.

requires a certain amount of time to take up its shrunken condition. The reserve of strength which remains after taking up the shrinkage stresses is usually adequate in the case of St. 37 but is very small for St. 52, and this accounts for the greater susceptibility of the latter to cracking.

Under external static loading, shrinkage stresses after welding are no more dangerous than other dead load stresses, but stresses which cause unstable equilibrium form an exception to this statement. Additional shrinkages due to neighbouring welds, or new internal stresses due to temperature effects, may lead to cracking, and this occurs more readily in St. 52 than in St. 37 on account of the smaller margin of stress and elongation possessed by the former.

It would be desirable to carry out fatigue tests using the experimental apparatus shown. Pre-stressing, in the sense the term is applied to ordinary fatigue tests, is present only if the loading acts in such a way as to counteract the shrinkage stresses. In such a case the fatigue strength must approximate to the elastic limit, and the available amplitude for butt welds, according to the fatigue tests carried out by the relevant German Commission,[1] amounts to 10 kg/mm² for St. 37 and 13.1 kg/mm^2 for St. 52. According to German regulations when $\gamma = 1$ this requires a value of α of approximately 0.65 for St. 37 and approximately 0.58 for St. 52. Where the shrinkage stresses are of the same sign, and usually also when they are of opposite sign, there can be only one distribution of stress in the member for which the fatigue strengths may be at a maximum but require to be determined in each case.

Thermal stresses still frequently offer difficulties to the engineer both in tha drawing office and in the workshop. It may be hoped, however, that research and experience may in the near future, lead to welding processes being so far perfected that shrinkage stresses will cease to offer any more difficulty than secondary stresses in rivetted work.

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[1] Dauerfestigkeitsversuche mit Schweißverbindungen (report of the commission on fatigue tests in the welding technical committee of the Verein Deutscher Ingenieure), p. 27 and 35—37.

IIIb 5

Temperature stresses observed in welded constructions in Belgium.

Bei Schweißarbeiten in Belgien festgestellte thermische Beanspruchungen.

Contraintes thermiques constatées lors des travaux soudés en Belgique.

A. Spoliansky,
Ingénieur des Constructions Civiles et Electricien A.I.Lg., Liége.

This note will be limited to a consideration of thermal stresses, regarding which it may be observed that although we do not know their exact magnitude we do know, only too well, that it is considerable. As a single example in support of this fact mention may be made of a Vierendeel girder wherein no special precautions had been taken, as they should have been, to relieve the thermal effects of the thick weld seams of 36 mm side: when the last site weld was being formed the end of the bridge was observed to be lifted off its bearing, and a fold appeared in the lower boom. In this instance the weight of the longitudinal member in question was approximately 80 tons.

Elastic action appears in the following forms:

1) Shortening of the members.
2) Deformations.
3) Internal stresses, which may or may not be attended by cracking and breakage.

1) *Shortening of members.*

Members are shortened as a result of shrinkage from welding, but by making the members slightly longer than necessary it is possible to ensure that the final dimensions are sufficiently accurate.

2) *Deformations.*

Deformations are more especially apt to be considerable at places where the weld seams are asymmetrical. Any given deformation is proportional to the free length of the member in question, where the latter is able to deform in the direction of the seam, and it is inversely proportional to the thickness of the member. Most of the welded bridges in Belgium are Vierendeel girders with a parabolic upper boom, and since this boom is in compression there is every advantage in making it as stiff as possible. There are two methods of doing so:

a) The use of double T girder having its flanges in the form of standard rolled joists 400 to 500 mm deep, or broad flanged beams (Fig. 1). Technically, from the point of view of welding, the second of these methods is open to the objection that the weld is formed on the web of the joist which is of limited thickness, so that the deformation is considerable. Moreover, most of the methods commonly used in the workshop for straightening bent pieces when cold are dangerous: for instance, in certain shops where joists used for booms of girders had been straightened cold, a series of cracks were found running at right angles to the weld in the web, due to a partial cold working effect on the metal (Fig. 3).

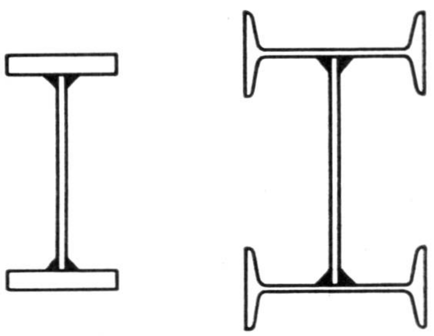

Fig. 1.
Types of booms for Vierendeel bridges.

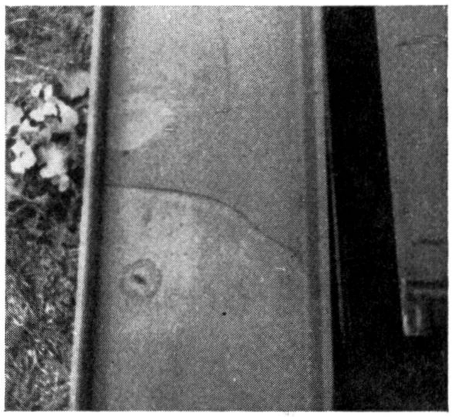

Fig. 2.
Fractured beam of an upper boom of a Vierendeel bridge.

The amount of these deformations can and should be reduced by suitable choice of

a) The thickness of the pieces to be joined, and

b) The sizes of electrodes to be used.

For this and other reasons, electrodes of large diameter should be avoided, especially for the first layers. In most cases deformations can practically be eliminated if a symmetrical arrangement of the beads is adopted.

In Vierendeel girders and rigid frames, such as are becoming general in Belgium, use is being made of connecting gussets having a section built up from a web with a flange (Fig. 3). Such gussets may be formed from four flanges b), with symmetrical beads, or with two flanges a). The arrangement shown at b) is evidently to be preferred, but it calls for a large number of welds. In box-shaped members (Fig. 4), notwithstanding the symmetry of the welds, a twisting effect has been observed, due partly to the amount of metal deposited in the different beads not being precisely equal and partly to the fact that the elements themselves are not precisely similar.

3) *Internal stresses.*

The shrinkage of welds gives rise to stresses extending over a considerable zone, and these may be dangerous especially where there already are pre-existing stresses. For instance, in rolled sections of great thickness the deposition of a weld bead may have the effect of causing fracture, and in Belgium breakages have often been found in Grey beams which, as is well known, are subject to

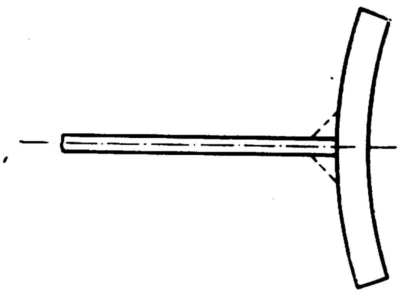

Fig. 3.
Inverted camber given to a welded plate.

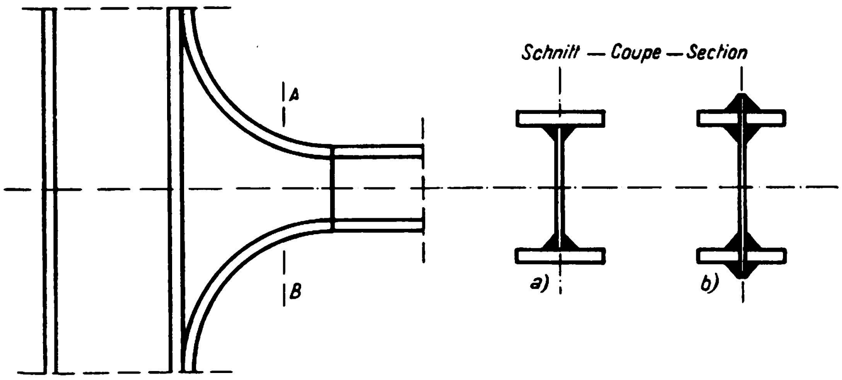

Fig. 4.
Details of panet point with tangential jointing.

heavy rolling stresses. The drilling of a hole may render a rolled piece unsuitable for welding because of the interruption of the cold worked zone, giving rise to cracks and breakage in the sound part of the piece. The crowding of many weld

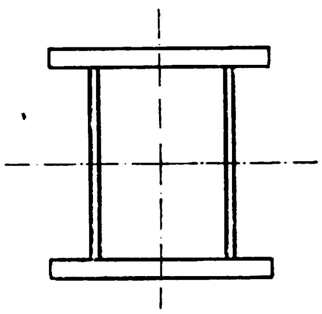

Fig. 5.
Welded box girder.

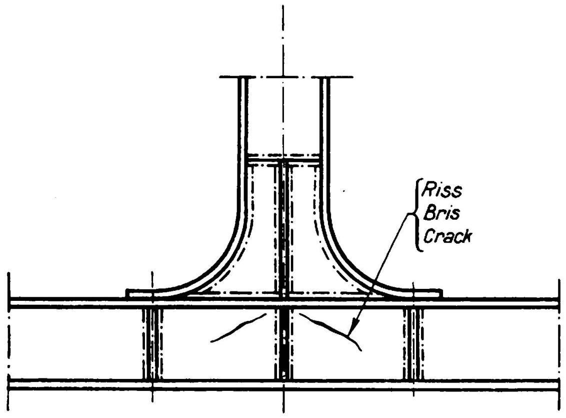

Fig. 6.
Details of panet point with characteristic cracks.

beads into a small space should be avoided, and in the same way it is desirable to avoid placing welds of widely different dimensions close together.

In Belgium there have been cases of accidents arising in the welding of the bracings to the intersections in Vierendeel girders with a number of stiffeners (Fig. 7), and also in the welding of the cruciform-sectioned verticals of these bridges (Fig. 8). In plate web girders the stiffeners are a source of great trouble, and if it were possible to diminish their number by suitably increasing the thickness of the web the design of such bridges would be improved. Another evident improvement in the design of stiffeners would be to continue them as far as the tensile boom (Fig. 9).

The chief danger attending the presence of these thermal forces is that any resulting cracks or breakages may not appear in the workshop immediately after the welding operation. Microscopic cracks may then be imperceptible, and escape detection until some months later: a form of delayed action which is peculiar to welding and which has never been completely explained. The effect may perhaps be similar to that sometimes observed in accidents to cast pieces where, likewise, the breakages have occurred at unexpected places.

An instance occurred in which the presence of a large number of cracks in the parent metal and weld metal was disclosed in the course of alterations to a girder of double T-section when a welded plate was being cut out with the blowpipe (Fig. 10); these cracks were probably due to will scale on the plate.

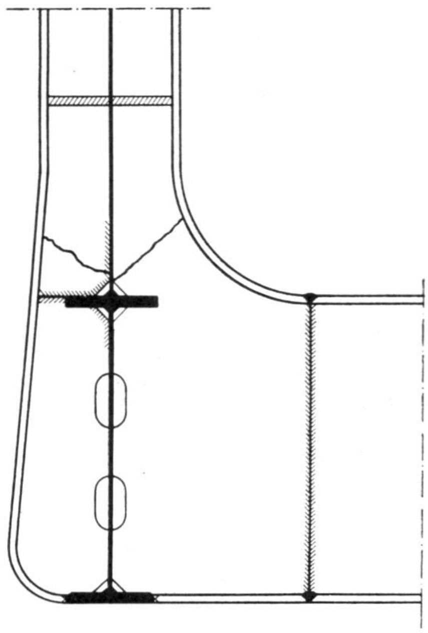

Fig. 7.
Characteristic cracks in uprights of Vierendeel bridges.

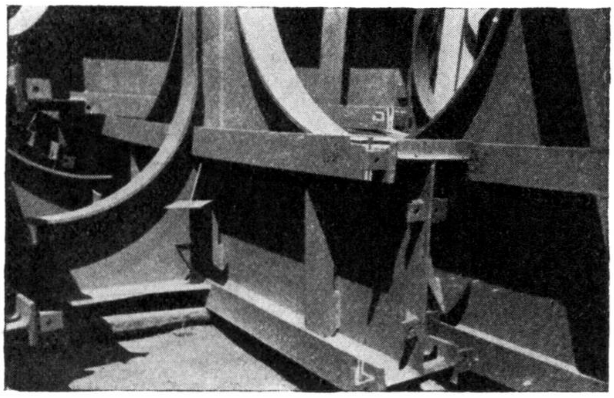

Fig. 8.
Connection of cross girder and standard.

Another example worthy of mention is the following: some months after the welding of a bridge had been completed and the concrete decking had been laid down a sudden breakage occurred along the axis of the welded joint of the web, accompanied by a noise (Fig. 11). In the author's opinion the shrinkage stress in the seams of the flanges, which were 36 mm thick, had been sufficient to initiate cracking in the bead along the web.

Precautions to be taken to reduce the effects of thermal stresses.

Advance precautions.

A) *Positions and dimensions of welds.*

Welds should be so placed as to receive a minimum of stress under all conditions, and their close proximity to one another should be avoided. The proper detailing of connections is a matter of the first importance to which, in Belgium, a great deal of consideration has been given and this has led to the perfecting of a curved form of joint with tangential connections, which is suitable for use both in bridges and in building frames. Mons. *Campus* has described this form of joint before the present Congress.

B) *Dimensions of the members to be joined.*

The thickness and length of the members must be carefully proportioned, and for the following reasons the thickness of plates should not be below a certain minimum:

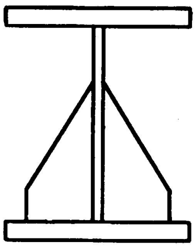

Fig. 9.
Stiffener to a single web beam.

1) It has been found that in thin sheets at temperatures of 600 to 800° C the elongation is considerably reduced, with a consequent increased tendency to cracking.
2) Excessive penetration of the welding is to be apprehended, and this, from an operating point of view, fixes a minimum thickness.
3) To minimise warping.
4) To reduce the number of stiffeners.

C) *Sequence of welding.*

The sequence in which the various beads are deposited should not be left to the decision of the welder, but should be laid down in a programme with a view to minimising the shrinkage of the welds, just as is done for the shrinkage of concrete work.

D) *Metallographic analysis of the parent metal and weld metal.*

This is a matter of the first importance, for it is necessary to be quite certain as to the weldability of the parent metal.

E) *Fabrication of members.*

This must be done with special care so as to avoid excessive tolerances which might call for an excessive amount of weld metal, with harmful effects.

F) *Special precautions.*

In special cases a particular procedure, such as pre-heating before welding, may need to be laid down.

Working arrangements.

G) *Electrical apparatus.*

Since it is important to be able to ensure a uniform deposition of weld metal great attention should be paid to the electrical installations and these should be specially designed for welding work. The transformers and cable leads must be such that drop in voltage and amperage is limited to a reasonable amount. A sufficient number of electric measuring instruments should be provided and continuously observed.

H) *Choice of electrical characteristics.*

This choice is a matter of great complexity. The temperature of deposition should be kept down in order to reduce thermal strains, but at the same time this temperature must be high enough to ensure good penetration. For any given jobthat is to say for steels of known composition when using electrodes of suitable type, where the pieces to be joined are of given thicknesses — there are certain optimum electrical characteristics which should be adopted.

I) *Limit of maximum diameter for the electrodes to be used.*

In Belgium a large proportion of the accidents which have occurred have been attributable mainly to the desire of the workshops to lessen their labour costs by working with too large a diameter of electrode. It is essential that this diameter should be kept down, in the first place on account of the danger which attends too rapid cooling and secondly because of the danger of using too heavy a current and causing correspondingly high thermal stresses. The maximum diameter of electrode has been provisionally fixed by the Ponts et Chaussées Belges at 5 mm, except for the bottom layer where the limit is 4 mm.

J) *Precautions to be taken in winter work.*

In Belgium it is now forbidden to carry out welding at temperatures below 4° C.

K) As a means of deciding the best method to adopt it would obviously be an advantage if the order of magnitude of the thermal stresses could be related to the various methods of procedure available, and in Belgium a group of engineers has entrusted a government laboratory with the task of making a complete survey of this subject.

Procedure after Operations.

L) The handling of pieces while still hot from welding should be forbidden.

M) Arrangements should be made to ensure that welds are allowed to cool slowly.

N) It is to be hoped that the experiments now embarked upon in Belgium will lead to the development of special devices which will be both practical and economical for lessening thermal stresses after welding.

IIIb 6

Recorded Failures of Electrically Welded Wrought Iron and Mild Steel Bridges.

Versager bei elektrisch geschweißten Brücken aus Schmiedeeisen und Flußstahl.

Ruptures enregistrées sur des ponts en fer forgé et en acier doux soudés électriquement.

H. J. L. Bruff,

Bridge Engineer, North Eastern Area, London & North Eastern Railway. York.

In connection with welding work on existing bridges, I have searched for but failed to discover any paper describing failures of welding, and as failures have been experienced in the carrying out of bridge welding by my Chief, Mr. *John Miller*, Engineer of the North Eastern Area of the London and North Eastern Railway, England, I submitted my paper "Recorded Failures of Electrically Welded Wrought Iron and Mild Steel Bridges" to this meeting in the hope that not only failures experienced by others might be brought to the attention of the International Association of Bridge and Structural Engineers, but that these failures as well as those described by me might be considered and their true nature and portent be determined, and if considered of sufficient importance, that the correct procedure to be followed and the necessary precautions to be taken to avoid similar failures be agreed on and laid down.

Since submitting my paper, failures somewhat different from those described by me have been recorded, and I am therefore taking this opportunity, so generously extended to me, to describe these.

The failures occurred in the course of carrying out the repairs to an old Wrought Iron plate girder bridge in the City of Leeds, which, apart from having suffered very badly from corrosion needed to be strengthened, as it was considered of inadequate strength to carry modern traffic.

When constructed in 1867, the local authorities insisted that the bridge should be given an ornamental appearance, as it was situated close to the chief centre of worship of the City and spanned the main approach to it. The main girders, of box construction, were therefore encased with ornamental C. I. plates in the then prevailing Gothic style of architecture, while the underside, until recently, was similarly boxed in with ornamental Gothic panelling. As a consequence, large portions of the bridge had remained inaccessible for painting and inspection since it was constructed, and when recently the casing plates were removed the ravages from corrosion were found to be serious and extensive.

There were two alternatives:

a) To renew the bridge, which would have been very expensive, as it was hemmed in on all sides with valuable business property, as a stoppage of traffic for even a short period was out of the question, it being one of the most important railway lines in the North of England.

b) To repair and strengthen the old bridge by electric welding, which method was adopted.

The observed failures of the welding occurred in the cross girders, which were of box type construction. They were badly corroded and had also suffered distortion of the webs due to overloading. This was caused by the removal, at some time, of certain rivets and their substitution with bolts which had rusted

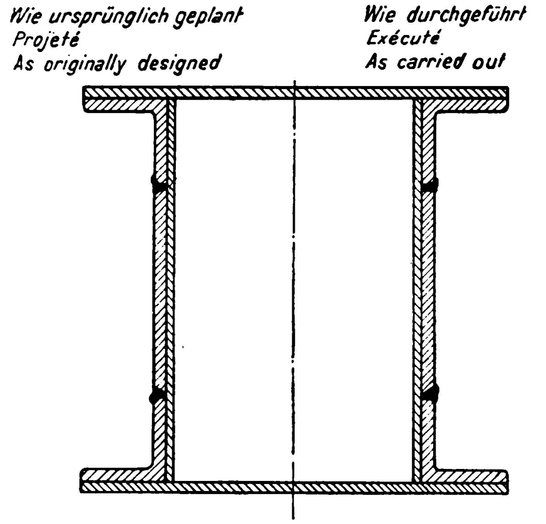

Fig. 1.
Section of Cross Girder.

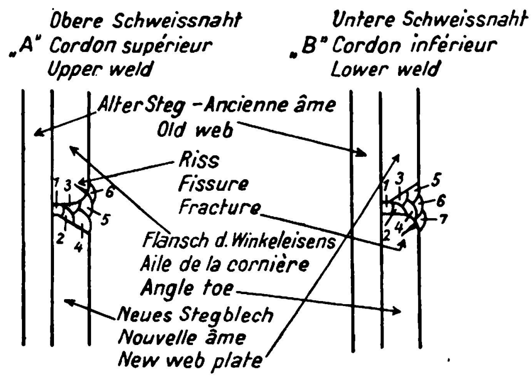

Fig. 2.
Details of Web Butt Welds as designed.

away. The work, therefore, became more extensive than anticipated, and of a very difficult nature.

The most difficult portion of the work was the welding in of new web plates, as these had to be fitted in between the vertical flanges of the cross girder angles, which necessitated using fairly large butt welds. The procedure of carrying out this work was to tack weld the new web plates to the angle toes and then to weld alternately the top and bottom butts in runs of six inches, the number of runs being in some cases as many as seven. The butts when prepared were of the usual form adopted by my Chief for welds of this kind and the welding was as indicated in Fig. 2, A and B.

It was found that when the final weld had been deposited, a fracture developed along the parallel to it, as shown on the photograph (Fig. 3). It was considered that the deposition of further welds (Fig. 2, A 6 and B 7) would not of necessity guarantee that these would reach sound metal at the root of the fractures.

After a number of experiments it was found that the method indicated in Fig. 4 was most satisfactory.

The lower or bottom weld was run first the full length of the web and then the upper or top weld. As will be noted, there was no chamfer in the case of the top weld, and in the case of the bottom weld, there was no face to the chamfer, which was run to a sharp edge. The sharp edged type of chamfer has

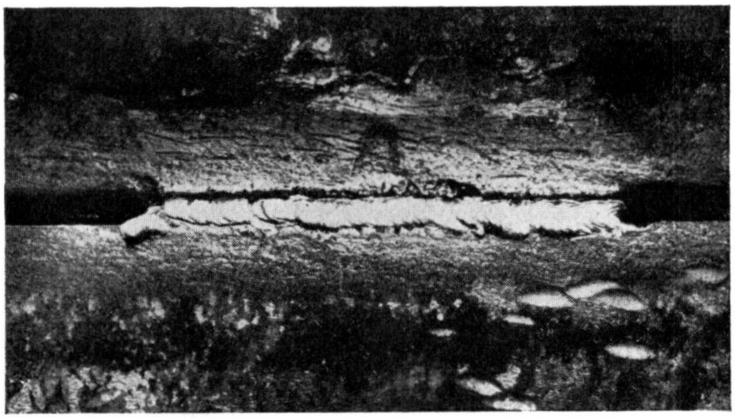

Fig. 3.

Fracture in angle parallel Weld.

since been adopted instead of the form shown in Fig. 2 above, as it secures better fusion at the junction of the plates.

The fractures seem to be caused by the contraction of the metal at right angles to the rolling direction extending as the welding proceeded. A piece of the

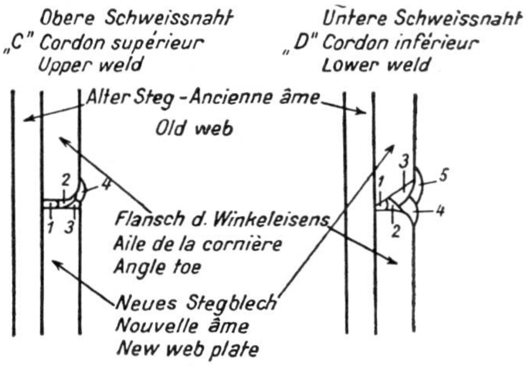

Fig. 4.

Web Butt Welds as carried out.

angle cut out disclosed a fibrous structure parallel with the length of the bar, such as wrought iron of good quality invariably shows.

The idea underlying the method finally adopted was to provide a coating weld (Fig. 4: C 2 and D 4) which would offer greater resistance to the contraction set up by the shrinkage of the welds (Fig. 4 : C 3 and C 4 and D 5) than the W. I. of the angles. After adopting the method described, no fractures have taken place when the welds were made, nor have any fractures developed since.

IIIb 7

The Reduction of Thermal Stresses in Welded Steelwork.

Verminderung der Wärmespannungen in geschweißten Stahlbauten.

La réduction des contraintes thermiques dans les constructions métalliques soudées.

Dr. Ing. A. Dörnen,
Dortmund-Derne.

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° 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—500⁰ 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.

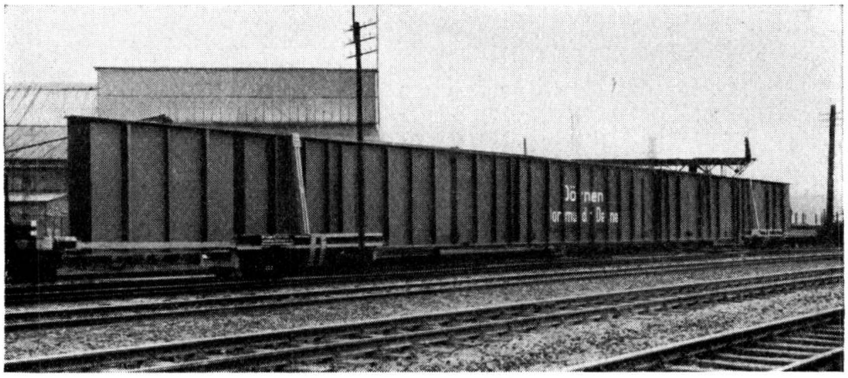

Fig. 1.

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

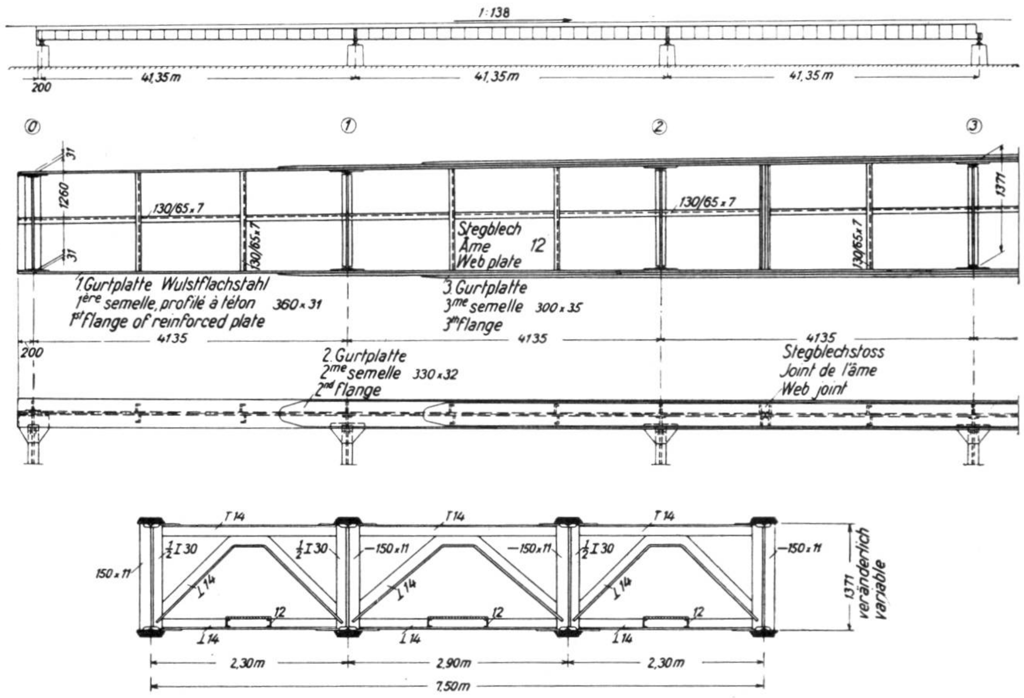

Fig. 2.

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

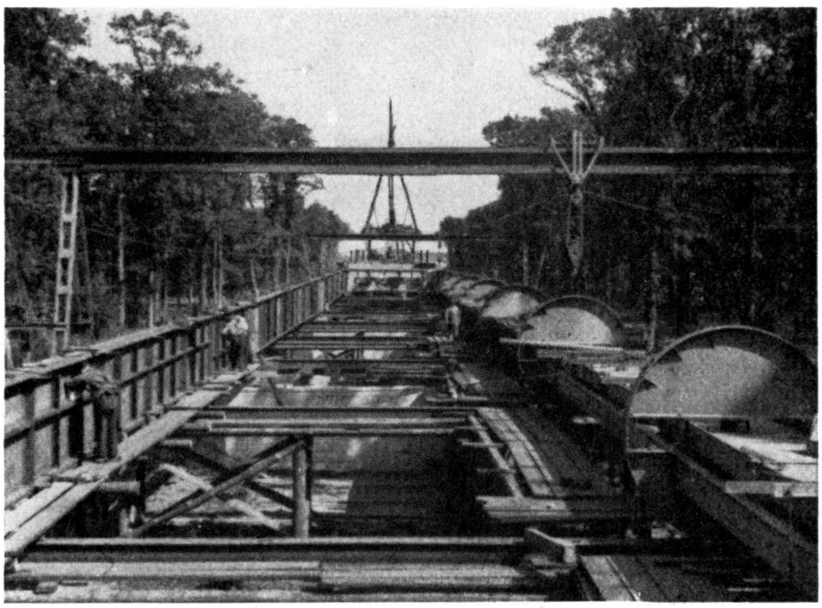

Fig. 3.

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-

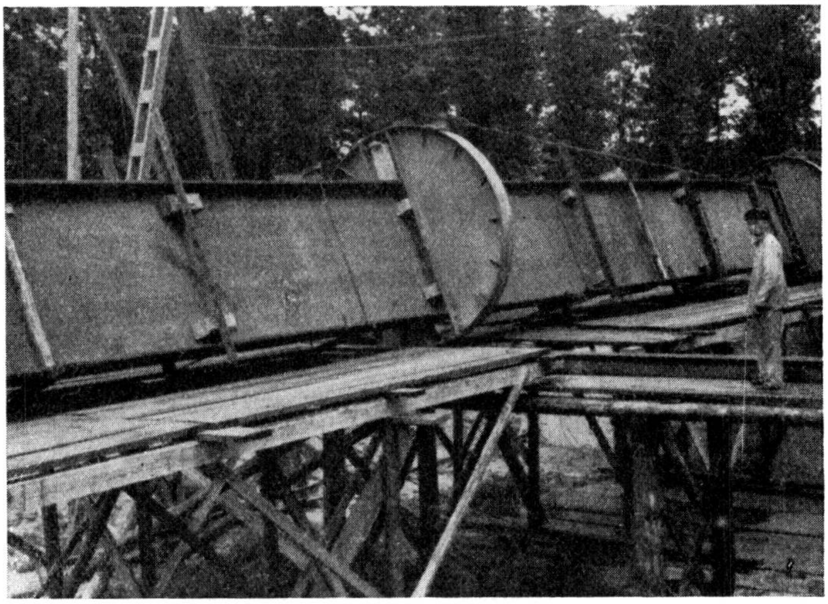

Fig. 4.

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

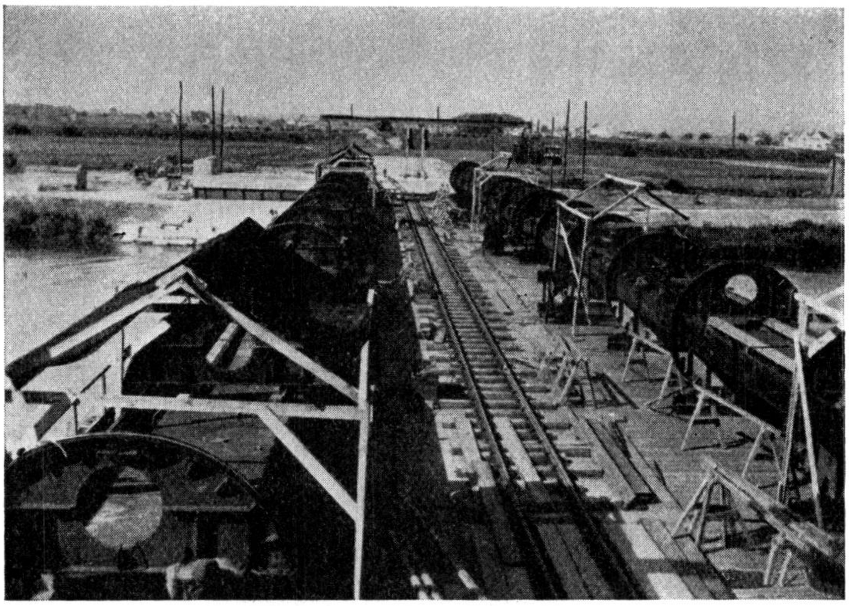

Fig. 5.

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

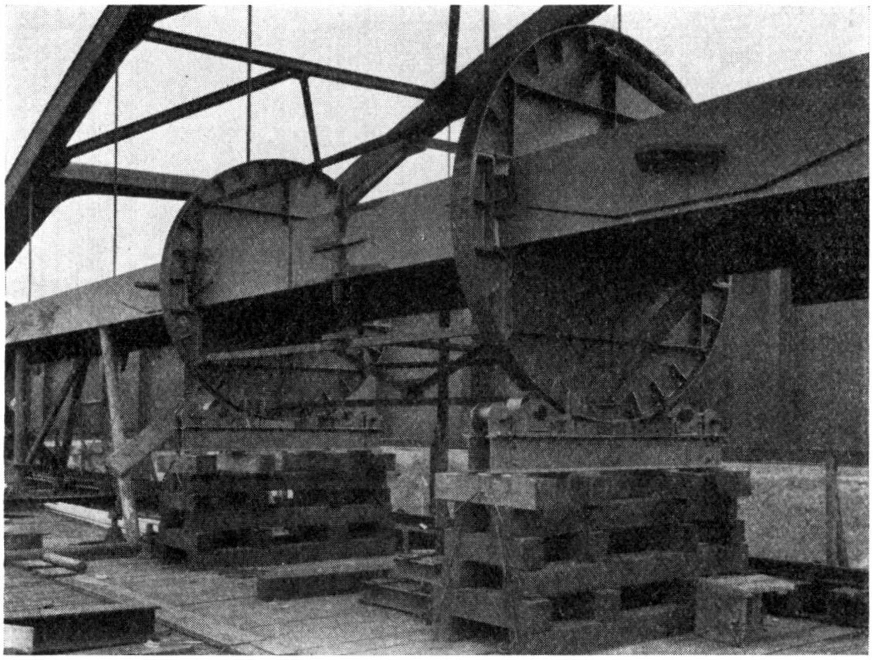

Fig. 6.

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

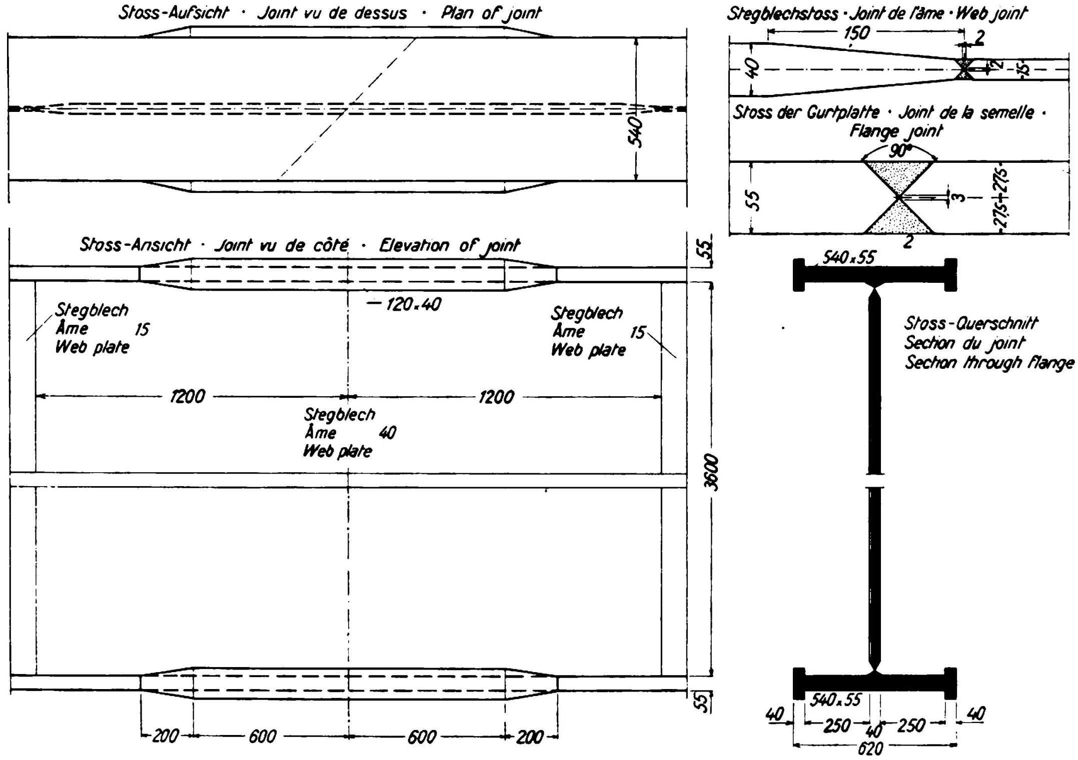

Fig. 7.

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

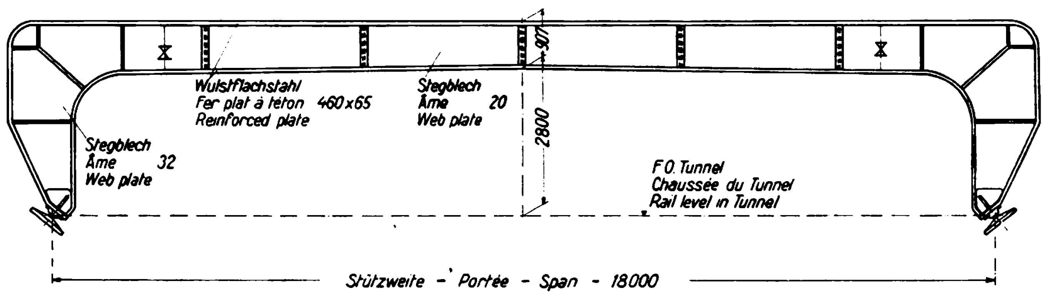

Fig. 8.

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.

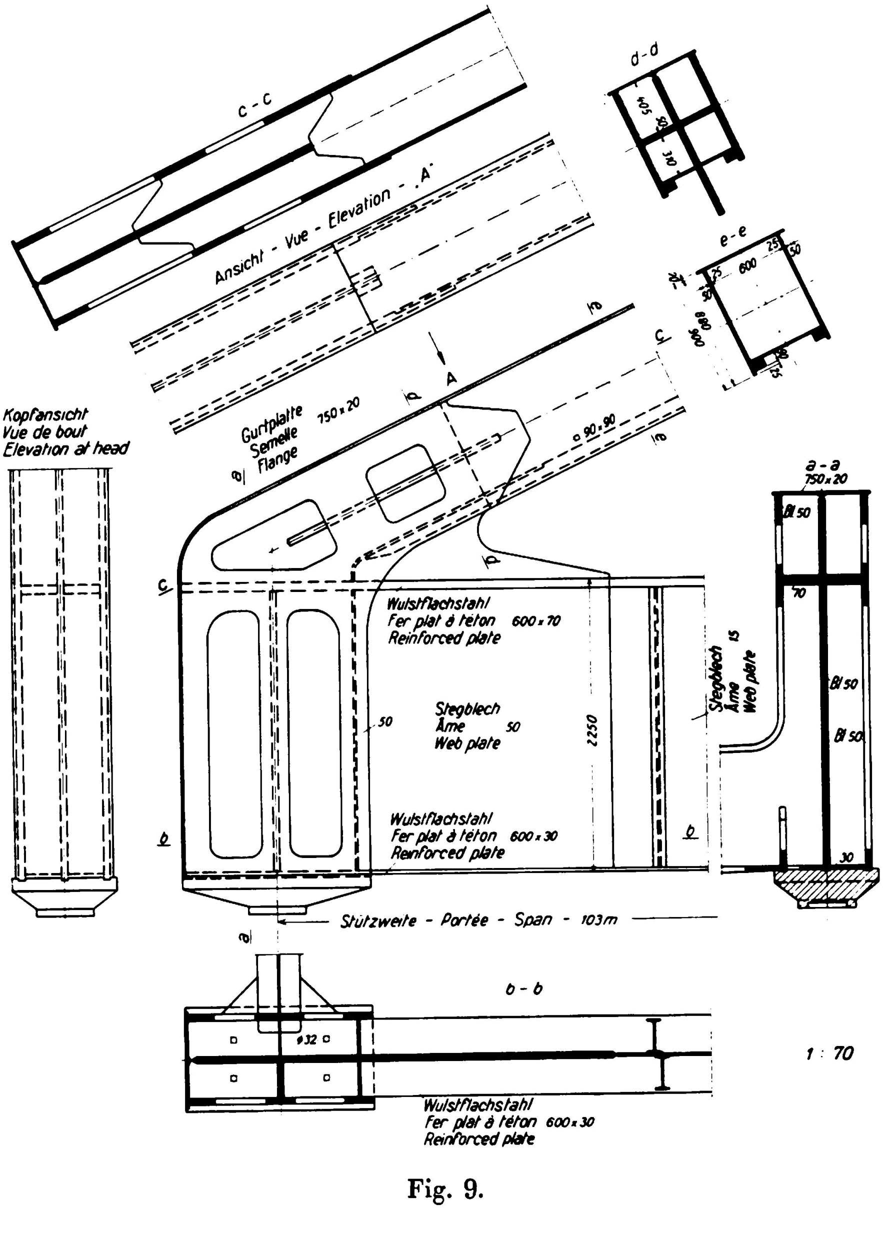

Fig. 9.

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

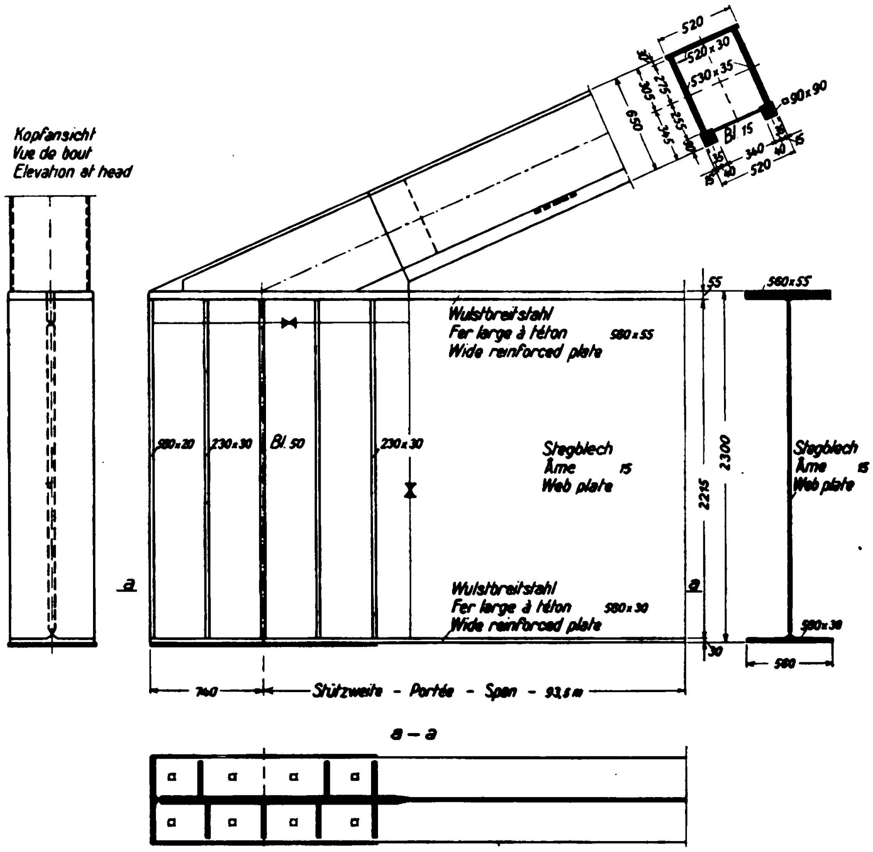

Fig. 10.

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.

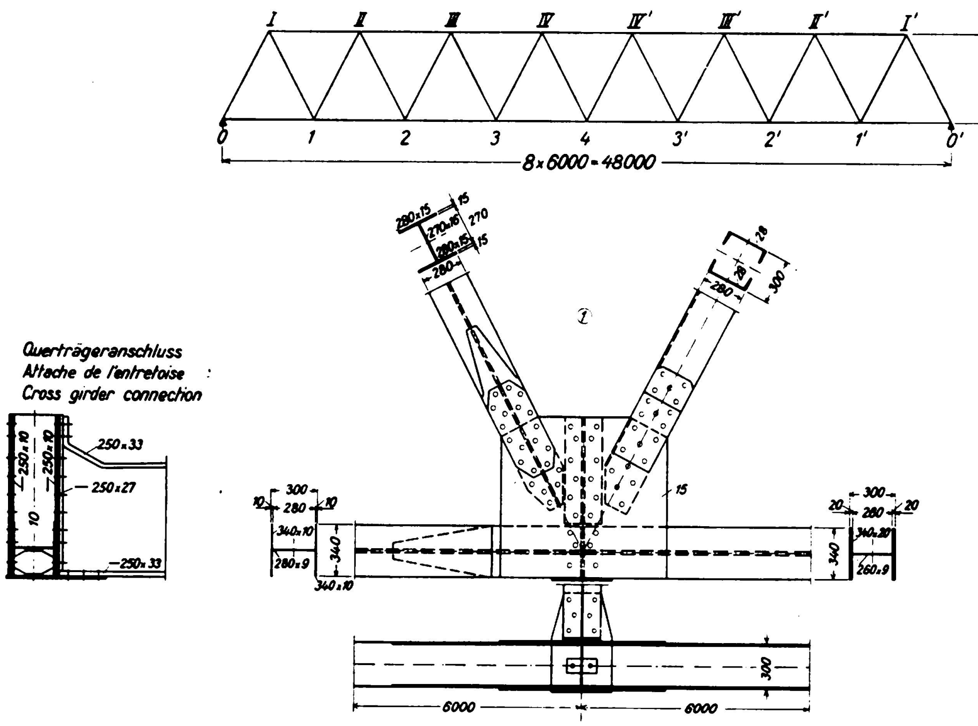

Fig. 11. Main girder. Intersection No. 1.

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*.

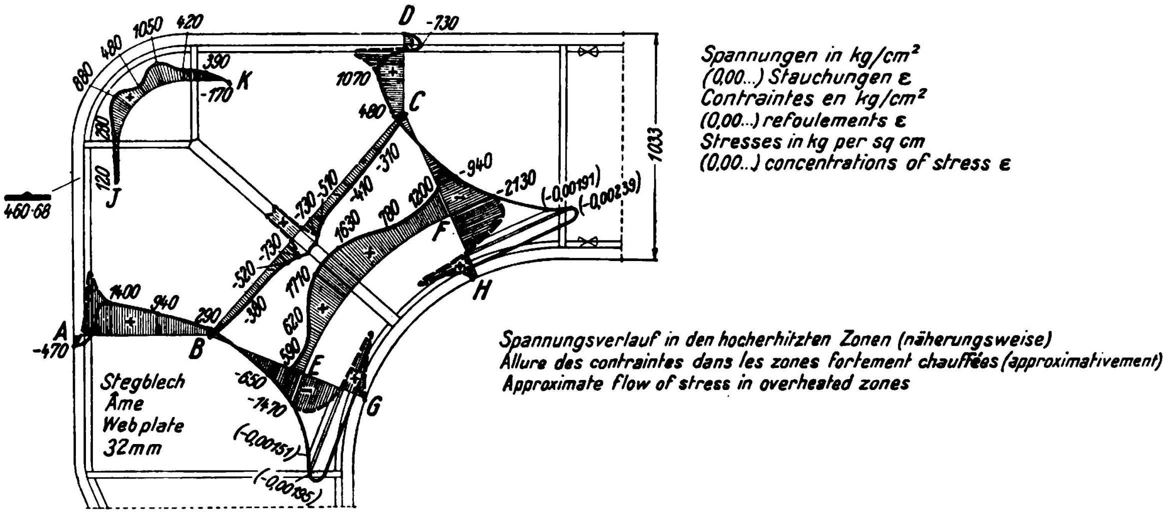

Fig. 12.

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.

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[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.

IIIb 8

Structural Welding in Practice.

Aus der Praxis der geschweißten Konstruktionen.

Sur la pratique des constructions soudées.

Dr. Ing. A. Fava,
Chef de Service aux Chemins de fer de l'Etat, Rome.

In Italy the use of welding has now become a matter of accepted practice in all branches of steel construction, but its application to bridges of large span still gives rise to notable difficulties. It is relatively easy, by adherence to the correct rules, to obtain welded joints which give the necessary strength even under the action of dynamic and repeated stresses, but difficulties arise as the result of the deformations which occur during the cooling of the seams in the construction of large bridges, and of the internal stresses which are thereby produced in the material. It has been found experimentally that these stresses may be very great.

If these deformations and the resulting internal stresses are to be avoided without the use of great thicknesses of material, it is necessary to adopt expedients and precautions which have the effect of considerably increasing the unit costs of welded work. Up to the present it has not been found, in Italy, that this increase in cost is compensated by a reduction in weight, so that under present conditions there is no economic advantage in using welding for large spans. The question is, however, receiving continued attention, in view of the emphasis that is being laid in Italy on the need to economise in steel, and it may be mentioned that preparatory work is now in hand among Italian steel makers for the production of special series of rolled sections which will facilitate welded work. These special sections will enable the deformations to be reduced, and it is anticipated that in this way an economic solution to the problem will be found.

On the State Railway system several bridges are in course of construction having plate-webbed main girders of spans up to 40 m which are completely welded. For still greater spans open-webbed bridges are preferred, wherein all the members including bracings are of welded construction, and only the site joints (including the connections between the web members and the booms) are rivetted.

Fig. 1 represents one of these bridges, and Fig. 2 shows one of its cross girders in course of being welded by means of an apparatus which enables it to be easily and rapidly moved about, so that the sequence of weld seams can be arranged with a view to minimising the tendency to warping.

Despite every precaution, however, the contraction which occurred in the cross section of the weld seams while cooling resulted in the bottom of the girders

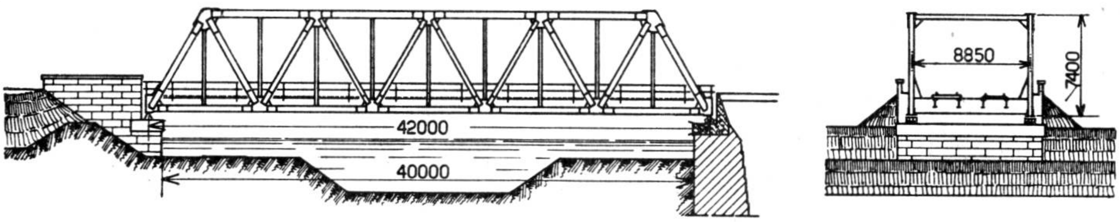

Fig. 1.

becoming bent, and at those points where (as for making the connections) it was essential that a perfectly flat undersurface should be obtained the only practicable remedy was found to be that of giving the plates to be used for the purpose a pre-imposed curvature in the opposite direction to that which would

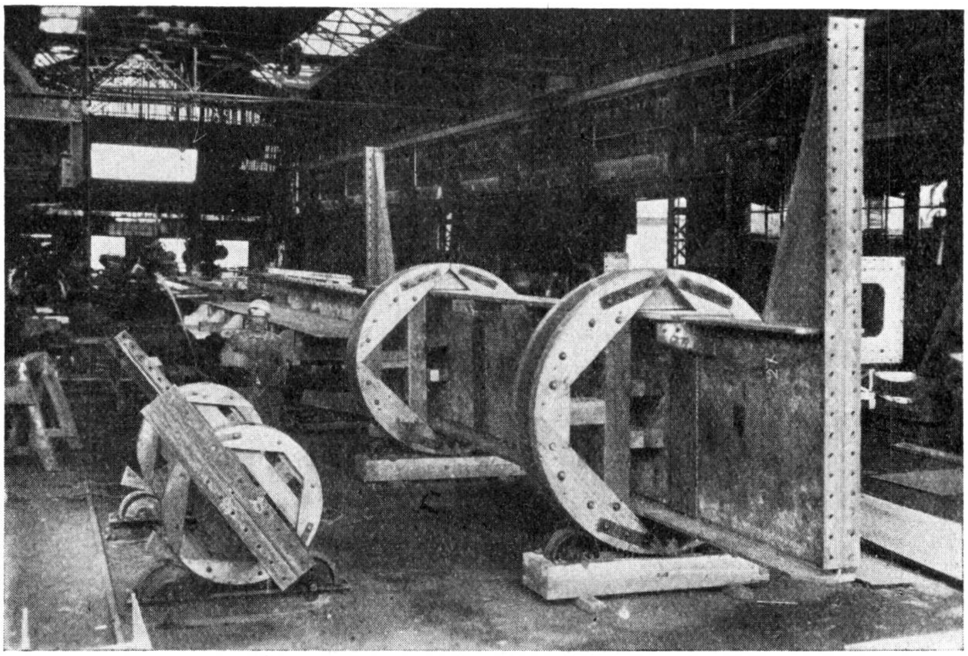

Fig. 2.

result from the cooling; an expedient which gives some idea of the costliness of the work.

Rather than enter into further details regarding these railway bridges, it is proposed to mention a few ordinary road bridges of completely welded construction, which, while of relatively limited size, may be of some interest as steps towards more ambitious work. These bridges have openwebbed main girders, and welding is used even for the connections of the web members. It was found possible to choose the sections of the various members in relation to the

Fig. 3.

loads to be imposed upon them in such a way as to minimise the tendency to deformation.

Figs. 3 and 4 refer to a number of bridges over the Isorno torrent near Domodossola, wherein both the boom and the web members of the main girders are

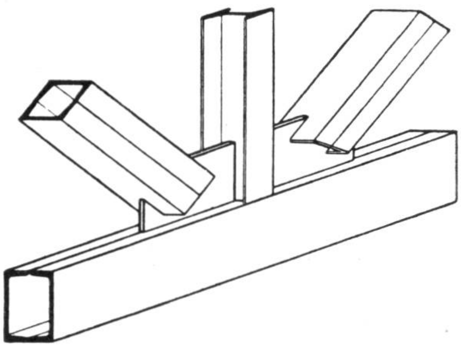

Fig. 4.

of tubular section built up by welding from two channel sections, slots being provided as in Fig. 4 for the insertion of connecting gussets.

Figs. 5 and 6 relate to two bridges of 25 m length over the Adige at Cengles. Here again the booms of the openwebbed main girders consist of double channel sections built up by welding. The diagonals consist of pairs of channel irons connected by plates. The verticals are slotted and were formed by the use of the cutting flame to make suitable openings in I beams, separating the two parts so

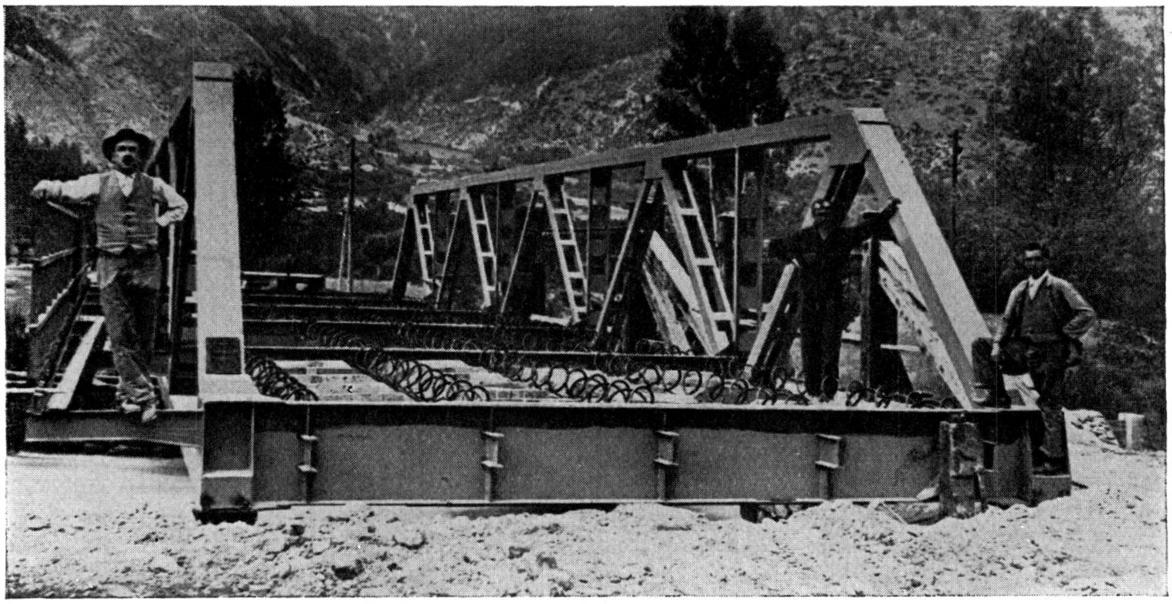

Fig. 5.

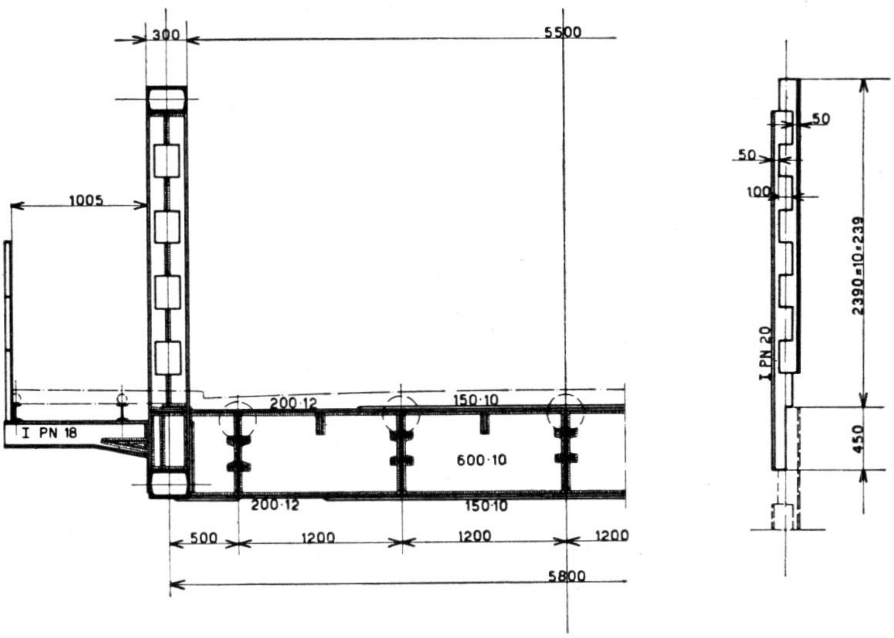

Fig. 6.

obtained and then welding the two projecting portions. The floor of the bridge consists of rolled beams of the well-known "Alpha" type, covered by a reinforced concrete slab.

Figs. 7 and 8 show a road bridge of 30 m span, in which the main girders are of solid-webbed construction and are reinforced above by a light arch in compression. The bridge gives a ligth and pleasing appearance and the whole of the booms are welded, including the and bearings.

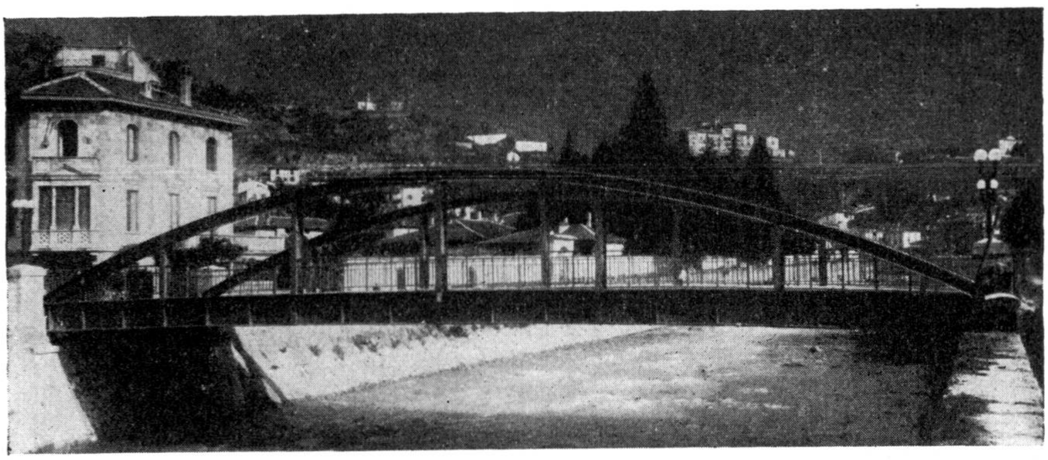

Fig. 7.

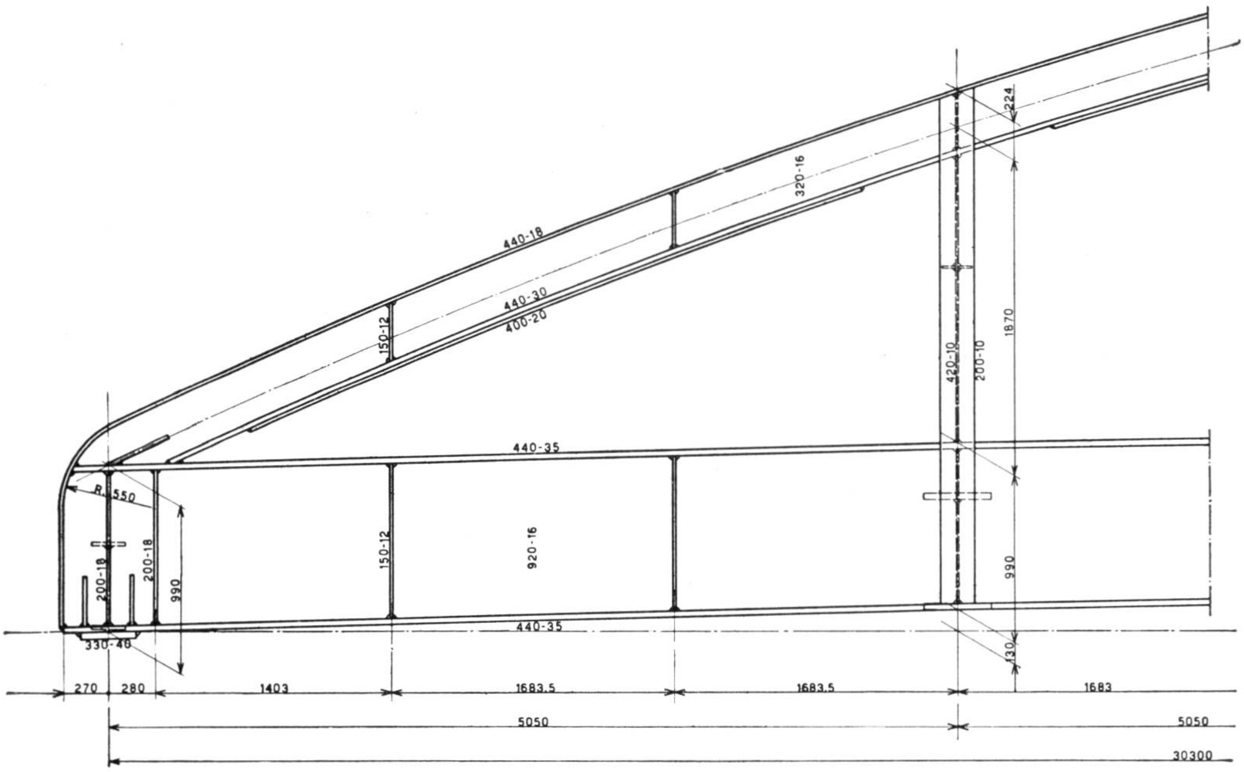

Fig. 8.

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III c

Inspection and control of welded joints.

Prüfung der Schweißnähte.

Contrôle des soudures.

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IIIc 1

Testing and Control in the Electric Welding of Ordinary Steels.

Prüfungs- und Überwachungsverfahren für die elektrische Schweißung der gewöhnlichen Stähle.

Méthodes d'essais et de contrôle de la soudure électrique des aciers ordinaires.

G. Moressée,

Ingénieur des Constructions Civiles et Coloniales, Liége.

Electric arc welding or resistance welding of those types of steel which are normally used in the construction of bridges and building frames is no longer considered a matter of any difficulty, but the economy, facility and aesthetic advantages of these methods are leading to their continued extension, and there arises a proportionately greater need to improve the methods of testing and controlling such work.

As a rule, specifications in regard to electric welding cover only the acceptance of electrodes, the control of the operatives and the examination of the beads deposited or of the spots welded. A conscientious designer, however, is under a grave legal and moral responsibility, which requires that he should be far more severe and exhaustive in his efforts so to control the work as to ensure success.

I. Electric arc welding.

Methods of use.

In Belgium the electric arc welding of mild steels (37—44 and 42—50 kg/mm²) and of semi-hard structural steels (St. 52, MS 60—70, C 58—65, etc.) is scarcely ever carried out otherwise than with protected electrodes of the coated or covered types. For load-bearing connections the use of bare wire electrodes has been given up on account of the brittleness and risk of oxidation of metal so deposited, which is due to the oxides and nitrides included in it (such as particles of SiO_2) or dissolved therein (FeO and nitrides up to 0,12 %).

Alternating current is generally employed in preference to direct current as the variations serve to provoke a violent agitation of the molten metal which eliminates slag and air bubbles. Its use requires that the arc should be kept short so as to ensure stability, and this produces large numbers of small drops of metal due to the concentration of the heat and the effective protection of the metal being melted. Heavy currents are especially sought after, provided the arc can be kept stable, as they allow the speed of welding to be considerably increased, thus reducing the cost and also minimising internal stresses; at the same time better penetration, and the desposition of a sound metal free from inclusions, are ensured.

Choice of electrodes.

The choice of suitable electrodes for any given job is a problem of the first importance, though too often left to the arbitrary decision of the commercial side. This choice depends on the nature of the steel to be welded, the type of connections (whether rigid, semi-elastic or elastic), the position of the stress-resisting element (butt, frontal, lateral, oblique or combined forms of weld), the position in space (horizontal, vertical or overhead seams), the place where the welding work is to be carried out (in the workshop or on the site), and even the atmospheric conditions.

The type of electrode having been decided upon, it is well to adopt the largest diameter compatible with the thickness of the pieces to be welded and the positions of the seams, for, given a suitable current density, the speed of welding increases directly with the efficiency of the materials and labour used and the internal stresses are correspondingly reduced.

Labour.

Good results depend largely on the quality and conscientiousness of the welding operators and of their supervisors. The health of the men should be checked by frequent medical inspections, and all arrangements should be made to ensure that the work is carried on under hygienic and comfortable conditions. At the beginning or end of every week, or at the most every fortnight, each welder should be required to make a sample plate (Fig. 1) which should be subjected to cold bending tests, and also cruciform specimens which must be found capable of fracture without damage to the seams, and the fracture should be examined macrographically (Fig. 2).

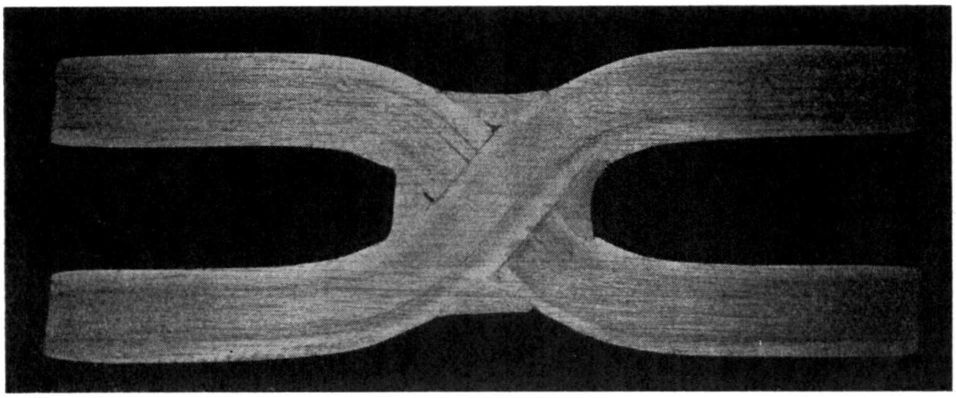
Weld metal.

The weld metal should be investigated as regards its chemical analysis (to afford a check on uniformity of quality), micrographical analysis of its structure (showing the influence of working methods on the size of grain, the penetration and presence of inclusions), macrographical analysis to show its condition of purity (inclusions, porosity, severe mechanical tests to indicate its physical characteristics (elongations at fracture being the criterion of quality, Figs. 3 and 4). See Figs. 5, 6, 7 and 8 which show an example of welding carried out on Ougrée carbon steel (58—65 kg/mm^2) using Arcos Stabilend electrodes.

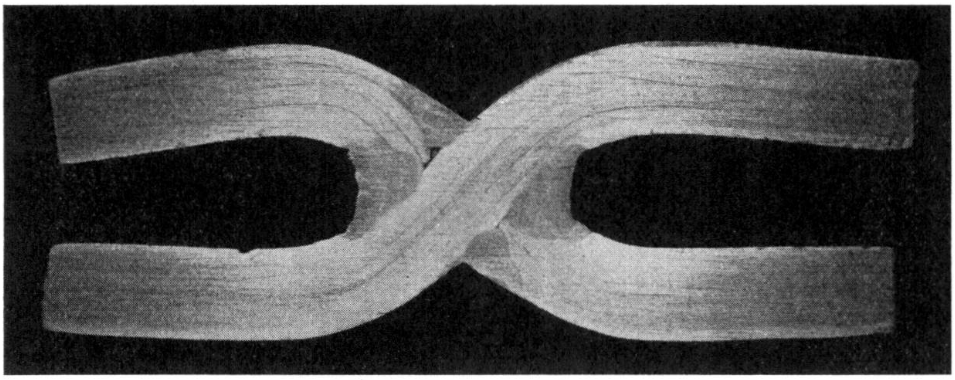
Welded connections.

The designer is concerned not only with the characteristics of the deposited metal, but also with the resistance of the welded joint as such to stress and to the corrosive agents in the atmosphere.

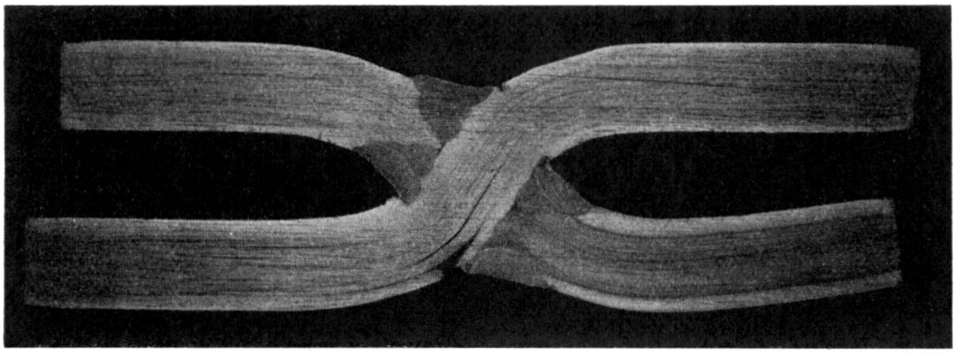
From this point of view specimens taken for the purpose of studying the weldability of a steel offer great advantages because, while approximating to the actual working conditions, they yield accurate results as regards behaviour in tension (including the determination of the elastic limits) and resistance to impact (Mesnager or Charpy resilience specimens).



Welds made with electrodes of 4 mm diameter and 190 amperes current.



Bevelled welds made with electrodes of 6 mm diameter and 250 amperes current.



Bevelled welds made with electrodes of 8 mm diameter and 500 amperes current.

Fig. 1.

Cruciform specimens: the seams must remain intact after crushing.

The next step is to check the strength of the different types of weld beads, adopted by the drawing-office under the conditions of execution which actually obtain, and under the conditions of stress which they will actually receive, and to study the nature of the strains which they will undergo after reaching their elastic

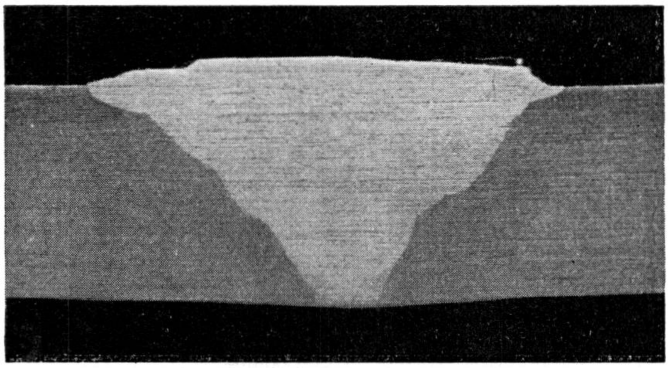


Fig. 2a.

Examination of a welded joint.

Chemical analysis.

Steel of 58—65 kg/mm² ultimate strength.		Metal deposited with „Arcos Stabilend" electrodes.	
C.	0.310	C.	0.080
Mn.	0.836	Mn.	0.430
P.	0.060	P.	0.014
Si.	0.075	Si.	0.015
S.	0.037	S.	0.032

Mechanical tests.

Breaking stress	58 kg/mm²	50 kg/mm²
Elastic limit	38 kg/mm²	36 kg/mm²
Elongation at fracture	24 %	28 %
Resilience	6 kgm/cm²	10.6 kgm/cm²

Average results of 80 tests.

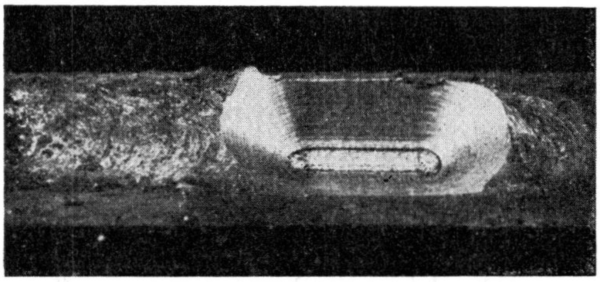


Fig. 2b.

Examinations by boring.

Fig. 2.

Examination of a specimen welded with alternating current using "Arcos Stabilend" electrodes; parent metal is of steel 58—65 kg/mm² ultimate strength.

Acceptance plate for steel of 37—44 kg/mm² (24 tests).

Breaking stress	min.	50 kg/mm²
	max.	56 kg/mm²
	average	54 kg/mm²
Resilience	min.	8.9 kgm/cm²
	max.	12.6 kgm/cm²
	average	11 kgm/cm²

Bending over mandril dia. = 3 · e, good at 180°.

Acceptance plate for steel of 58—65 kg/mm² (24 tests).

Breaking stress	min.	55 kg/mm²
	max.	69 kg/mm²
	average	58.5 kg/mm²
Resilience	min.	5.52 kgm/cm²
	max.	8.27 kgm/cm²
	average	6.7 kgm/cm²

Bending over mandril dia. = 5 · e, good at 180°.

Fig. 3.

	Without heat treatment.	After normalising treatment.
Weld metal.		
Weld joint.		
Close to joint.		
Raw metal.		

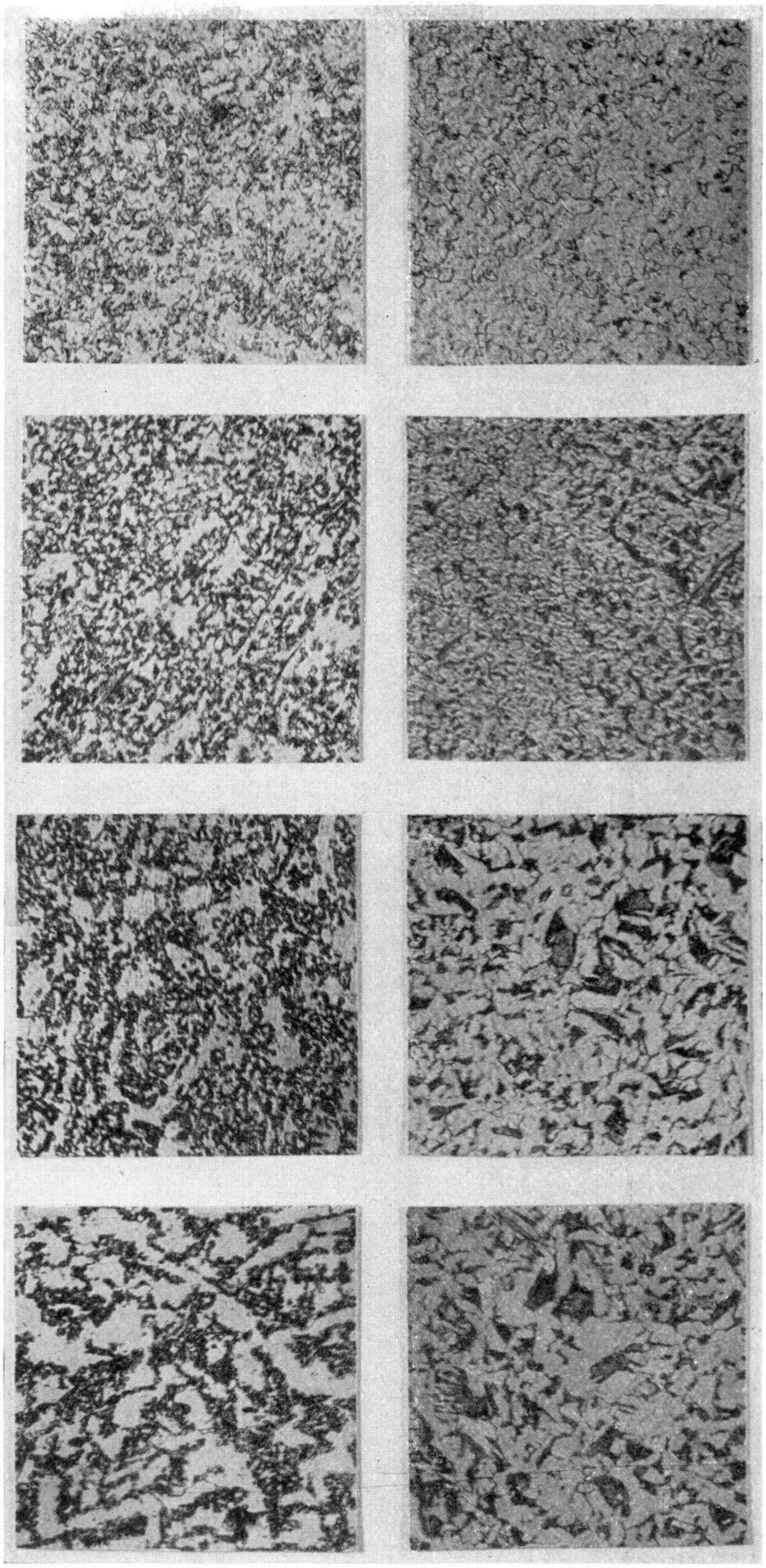


Fig. 4.

Micrographical study of a welded joint.
Carbon steel of 58—65 kg/mm². Electrode: "Arcos Stabilend".

limit. The distribution of the stresses may be studied by the tensometric method, either on a small scale model or on a model of the size adopted by Professor *Campus* for investigating the welded intersections of the frameworks for the Institut de Chimie et de Génie Civil at the University of Liége.

The first check is afforded by visual inspection of the weld beads on the part of a specialised supervisor. The welds are stamped by the welder with an identi-

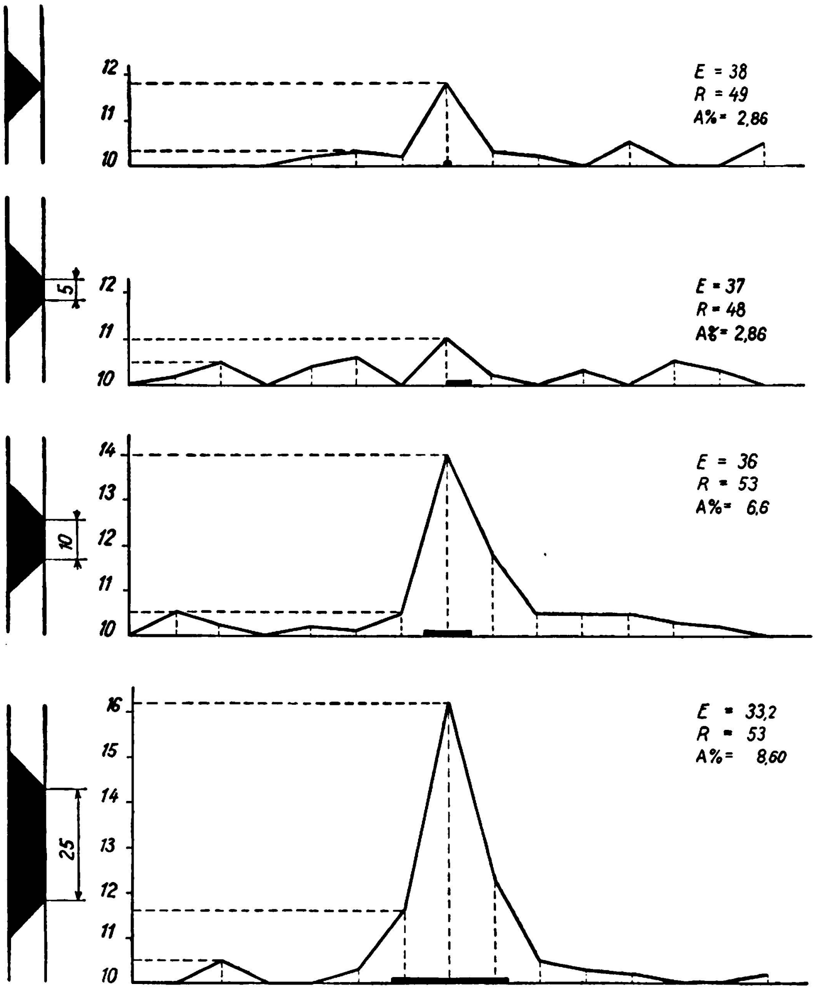


Fig. 5.

Tensile tests on flat specimens of 100 mm² cross section, 140 mm long, with varying amounts of weld, measurements of elongation being made at every 10 mm. Carbon steel 58—65 kg/mm². Arcos-Stabilend electrodes. The distribution of the elongation, and the smallness of its total amount, should be noticed. A control specimen cut from plate gave 20 %.

fying number, which makes him responsible for his work and enables defects to be traced to him, even if they do not appear until a long time afterwards. A record of these marks is kept in a special register with all relevant information.

Borings made into the weld seams with a drill may be made to yield useful information at the beginning of a job for the purpose of checking penetration, etc., but at later stages this destructive method possesses no more than a secondary interest due to its moral effect on the workman who knows that his work is being checked in this way.

The use of the stethoscope is not effective in the applications here considered.

Control by means of the magnetograph can be made to yield very valuable information in special cases, particularly in the advance examination of butt welded specimens. The method consists in using a magnet to produce a magnetic field and in studying the deviations imposed on the lines of force by the presence of the weld or in measuring the variations of this field. Any increase in the magnetic resistance of the weld due to faulty zones (cracks, air bubbles, coagulations), is made apparent in the lines of force of the spectrum.

In examining pieces of small dimensions remarkable results may be obtained with the Giraudi patent "metalloscope". Here the intensity of the field has to be made so great as to bring it to saturation, and a liquid which carries magnetic metallic oxides in suspension is poured over the pieces, becoming attracted to those points where the flux is dispersed in the surrounding air. In this way a pattern is projected on the surface which corresponds to any defects existing below.

The radio-metallographic method of control, which is the only effective one to be made practicable up to the present time, consists of photographing the weld lines with the aid of X-rays. The intensity of the rays must be regulated as a function of the thickness of the pieces and of the dimensions of the defects to be detected. The Phillips "Metallix" macroradiographic apparatus, either fixed or mounted on a motor trailer, gives a penetration of the X-rays amounting to 80 mm through steel. The electronic intensity is regulated by a rheostat, and complete protection is afforded to the operator (Figs. 9, 10, 11 and 12).

II. Spot welding.

Principles of use.

Electric resistance welding by the spot system is at present scarcely applied to bridges and building frames, except in the construction of beams, grillages and secondary members, but its use is continually extending.

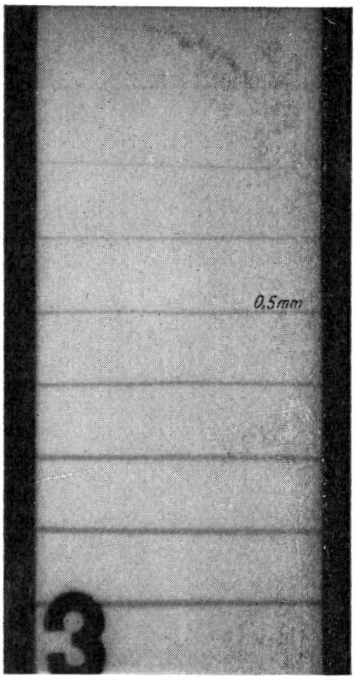
The proper execution of a spot weld depends on three conditions: the temperature of the welded zone, the time taken and the pressure exerted on the pieces to be joined during and after the operation.

It is difficult to measure the temperature of the metal in the neighbourhood of the weld directly. The time of welding has to be extremely small ($^1/_{50}$ of a second for rustless steels "18/8", with a view to the avoidance of certain chemical transformations in the metal) and it has to be varied according to the condition of the pieces. The pressure is kept constant for a given case and is regulated by mechanical, pneumatic, hydraulic or electrical means.

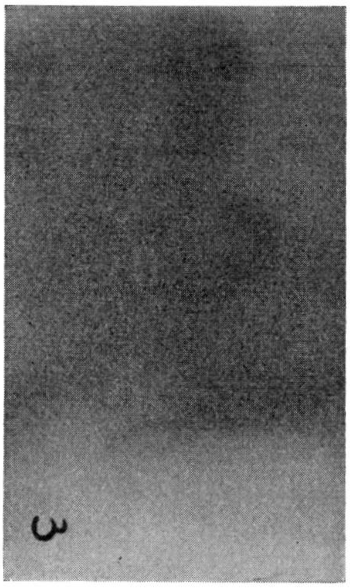
The best results are obtained by using very heavy pressures during the short time that the circuit is closed, and automatically releasing them as soon as the current is interrupted.

Control.

Excellent results are obtained through control of the weld by suitable switches designed to break the current at the proper time. Constant time switches will not always compensate for irregularities when the resistance varies in accordance with the state of oxidation of the pieces and with variations in starting, for their

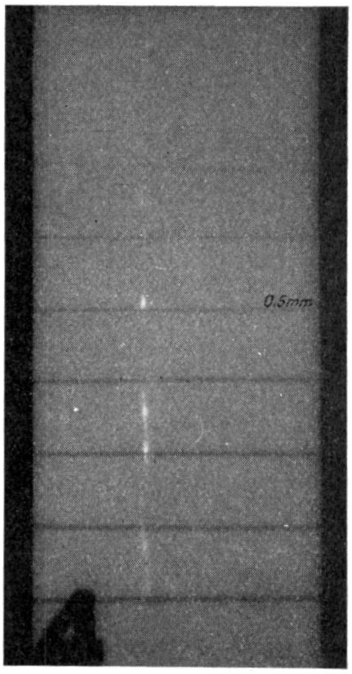


X-ray photograph.
(Exposure: 10 minutes. 70 KV. 4 m A.
Detection of faults down to 0.1 mm).

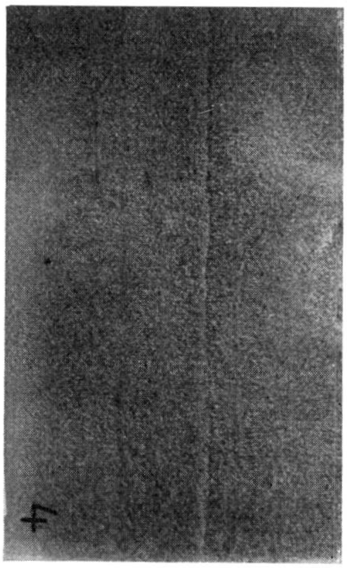


Magnetic spectrum.

Fig. 6.
Welded V-joint. Thickness 10 mm. Electrodes Esab. O.K. 47. Mild steel.
Sound weld.

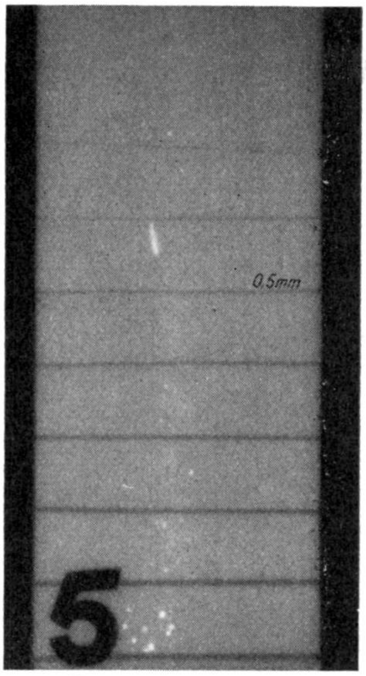


X-ray photograph.
(Exposure: 15 minutes. 70 KV. 4 m A.)

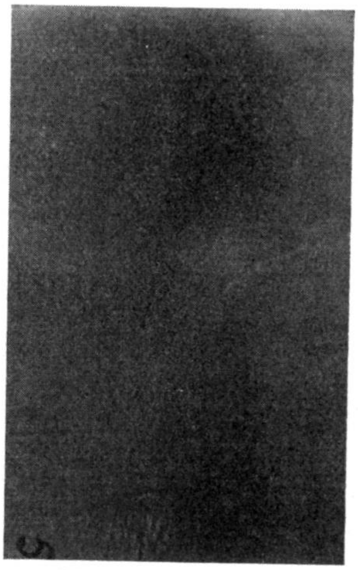


Magnetic spectrum.

Fig. 7.
Welded V-joint. Thickness 10 mm. Electrodes: Esab. O.K. 47.
Carbon steel of 58—65 kg/mm^2 ultimate strength. A longitudinal shrinkage crack will be noticed.
Externally the weld appeared perfect.



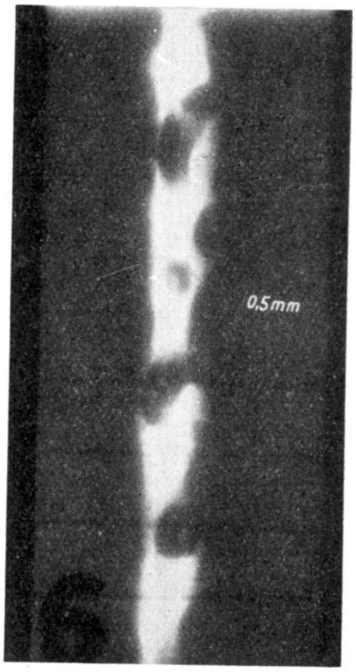
X-ray photograph.
(Exposure: 15 minutes. 70 KV. 4 m A.)



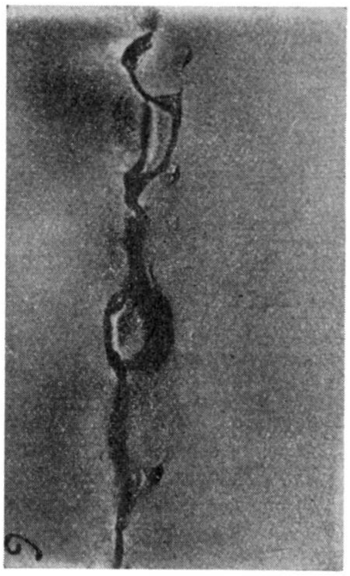
Magnetic spectrum.

Fig. 8.

Welded V-joint. Thickness 10 mm. Electrodes: Arcos-Veloxend.
Pores of the order of 0.1 to 0.2 mm due to excessive strength of current.



X-ray photograph.
(Detection of faults down to > 0.3 mm.
Exposure: 4 seconds. 90 KV. 4 m A.)



Magnetic spectrum.

Fig. 9.

Welded V-joint. Thickness 10 mm. Electrodes: Arcos-Veloxend.
A void will be noticed running the whole length of the seam, with drops of metal enclosed in slag.

efficiency depends on the assumption that the welding temperature will remain constant over two successive operations, and this is not the case unless the induced electromotive force in the secondary remains constant, and unless the sum of the contact resistances between the work and the electrodes is the same at every instant. The latter condition does not hold good unless the plates are absolutely clean, so as to offer both a constant striking resistance and a constant voltage drop.

Constant-minimum-current switches operate by cutting of the current as soon as its intensity passing through the weld reaches a pre-determined value. Their adjustment is delicate and they are at the mercy of the supply voltage.

Ampere-second integrating switches work by cutting of the supply current as soon as a pre-determined total of ampere-second has passed through the machine. It has been found that the ratio between the current passing through the transformer and the energy put into the weld varies constantly, and depends essentially on the surface condition of the pieces, on the pressure, and on the variation of the latter during the welding operation.

Watt-meter switches serve to measure the correct amount of power supplied to the machine, or some function of this power. They are preferably included in the secondary circuit at the electrodes, since in that way they give more precise indications, but they necessitate a current transformer and a complicated form of connection.

A recording alarm controller is available which makes an arc on a ribbon of paper to represent a function of the power supplied in welding each spot, which serves as a criterion of quality. In case of any accident, bad contact, badly connected conductors, faulty regulations, drop in voltage, etc. an alarm bell rings which attracts the attention of the operator, and the welding machine automatically stops until the conditions are put right.

Only clean pieces of work, as far as possible milled or sanded, should be used for welding.

The welded spots.

The welded spots should be examined in the laboratory by micrographical analysis, especially where steels containing copper, nickel or chromium are being

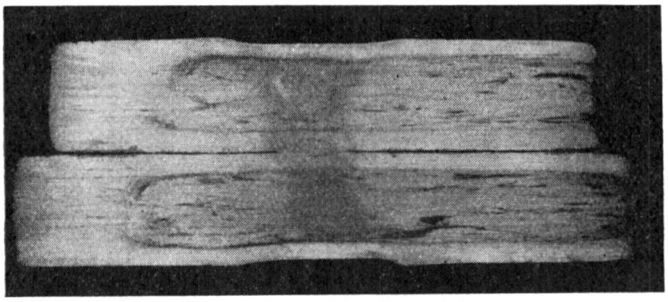


Fig. 10a.

Macro-photograph of a spot weld. Steel 42—50 kg/mm^2.

used. Any impurities which may be present in the contact zone remain embedded in the molten metal and become concentrated (Fig. 10).

Preliminary punching tests are necessary, and uniformity of results may be assured by using the modern forms of apparatus provided with watt-metric switches.

Raw metal. Metal in welded zone.

Fig. 10 b.

Micro-photographic study of a spot weld. Steel 42—50 kg/mm^2.

As the time of welding is always relatively short and consequently the cost per spot is small, a large number of spots are used ,so as to be able to work with a low working stress. The indication drawn from practice is that for thicknesses of up to about twice 10 mm the reliability of welding is of the same order as that of the corresponding rivetted work.

IIIc 2

The Testing of Welded Bridges and Structures.

Prüfung der geschweißten Brücken und Hochbauten.

Contrôle des ponts et charpentes soudés.

F. Campus,
Professeur à l'Université de Liége, Directeur du Laboratoire d'Essais du Génie Civil.

In the case of welded structures the necessity for control is not limited to the quality of the welds themselves, but includes closer supervision of the nature and treatment of the steel than is usual in rivetted work. This is a consideration to which, perhaps, not enough attention has been paid, and it is the clue to many difficulties and failures.

Here the author has in mind not only the quality of the steel as defined by the ordinary tests, but the fact that the steel must be subjected to tests of weldability — tests which are metallographic as well as mechanical, and which are designed to ensure the best possible connection between a given parent metal and a given weld metal. Even at this early stage it is necessary that due account should be taken of the special characteristics of the structure to be built.

The special shapes in which members, made from steel described as weldable, are incorporated in a job demand careful consideration. Welded construction has not overlooked the possible advantages of using rolled pieces, such as joists or flats, in unusually large dimensions or thicknesses. Now the metallurgical production of these pieces is a matter of some nicety, and the thermal and mechanical operations which they undergo during their manufacture may confer upon them unknown properties, resulting in an individuality which may be bad, or open to abuse. The subsequent welding of these pieces may influence them unfavourably. Moreover, welded members of this kind are frequently made the object of considerable preparatory work, such as cutting with the blow-pipe into shapes which are often complicated, and these operations may be enough to affect them adversely, even before welding is begun. If the defects are serious they will be detected and the piece will be scrapped, but what is more disturbing is the fact that a small defect may admit of being repaired or hidden. Within the range of the defects liable to arise before or after welding some may exist which are invisible, such as undetected cracks — for how otherwise can one account for the formation of visible cracks not appearing until long after the structure has been tested and put into service? Such cracks may not appear under load, and may show no sign of being the result of fatigue, but may be entirely of the kind here contemplated. As a rule, moreover, such cracks do not appear instantaneously, but only after a varying period of time has elapsed since the treatment which gives rise to them.

In the cases that have come to the author's notice it has been the exception for such a crack to occur in the weld itself. Generally speaking they appear in the parent metal, and though it might be supposed that the control exercised over the quality of the welding would exclude such a possibility, they may make their appearance many months after acceptance of the material.

It will be seen, then, that welded construction necessitates qualities in the pieces to be connected as definite as those characterising the methods used for connecting them. The process of manufacturing the steel, the method of rolling, the dimensions of the pieces, any subsequent heat treatment: all these points are as important to the structure as the quality of the welds. The same is true of the methods of fabrication, such as shearing, cutting, drilling, etc. Many instances of cracking — sometimes long delayed — have been the result of these operations, especially where they have given rise to origins of cracks. Wherever possible, therefore, drilling should be preferred to punching, sawing or hot cutting, to shearing etc. At least the ragged edges left by cutting should be milled or ground away, punched holes should be reamered out, etc.: in fact the metal should be nursed to the utmost extent that is economically possible. Finally, the sizes and shapes of the elements, their shaping and tooling, are all matters closely connected with the character of the structure as a whole; the preconditions of safety and control are determined, in their respective importance, by the design of the job as a whole. The use of welding in bridges and steel frames introduces complexities which amount to a revolution comparable with that brought about in masonry construction by the introduction of reinforced concrete.

Indeed, this analogy with the peculiarities of reinforced concrete holds good from more than one point of view. So far as the question of control is concerned, it serves very well to illustrate the distinction between control over the quality of the welds and control over the welded structure. Here the relatively longer experience of reinforced concrete practice is of value as a guard against illusions and exaggerations: for a long time past control over the quality of cement and concrete has been practised, but control over reinforced concrete structures is a more complicated matter, in reference to which it would be possible to paraphrase nearly everything which has been stated above regarding welded structures. Present practice in the control of reinforced concrete structures may usefully serve as a guide to the development, as well as moderating the requirements, of control over welded structures.

When all is said, control over the quality of the welds remains a primary element in the safety of bridges and frames using this method of connection. The available methods have been pointed out in the first part of this paper, but their practical application to bridges and frames is exposed to various difficulties which arise from the complication of these structures, and indeed amount, in some cases, to impossibilities. It has to be admitted that welding as applied to bridges and frames cannot be made the subject of so perfect a control as is applied to simpler work, such as tanks and pieces of moderate size which are mass-produced, rail joints, certain special mechanical constructions, and the like. This enumeration suggests, moreover, that a control of the kind here envisaged is not actually a necessity in large structures: or, if it is preferred to express the matter in another way, the economics of welding as applied to bridges and frames should be based on the idea that control is not absolute, but is relative and imperfect. Wisdom consists, then, in seeking and obtaining safety notwithstanding such imperfection; and this is perfectly feasible. This attitude is one which necessitates a special study of structural forms and connections; and it may be anticipated that the least reliable element in the structure as a whole will not be the welding, despite all the admitted imperfection of the latter. The remark, already made, as to the greater frequency with which cracks are found in the parent metal than in the weld metal lends weight to this principle and provides its practical justification. The principle may, indeed, be enunciated in a concise form by the dictum that fracture must never occur in the joint.. This is a condition which is perfectly feasible, and which in well-designed structures is in fact realised even under dynamic tests, a result which cannot be obtained by rivetting.

If this conception of the matter is adequate — an assumption which time must decide, and which is at least provisionally acceptable — it will still be useful and necessary to impose the most stringent possible guarantees on the quality of the welding, by adopting a system not very distantly allied to that which is practised in reinforced concrete work, and which indeed, is even now capable of higher accuracy than the latter.

Such methods are in fact already generally applied, differing as between one country and another, or between one job and another, only in details. They consist in a series of precautions laid down in specifications or regulations with the intention of giving guarantees, which shall for practical purposes be effective and adequate. For the sake of discussion the author proposes here to summarise the conditions applied to structures erected under his charge in Belgium in 1932, 1933 and 1934; conditions based on principles contained in a specification published at the beginning of 1932 which was the first official document of the kind in Belgium, from which the regulations, as finally published in Belgium, differ scarcely at all.

The fundamental conditions rest upon the principle governing the safety of joints as stated above, and these are followed by measures of control over the quality of the weld metal. On the hypothesis (which was true of the case under consideration) that special steel is to be used, this control must at the same time serve as an actual test of weldability. The steels employed were of the grade 42/50 (Belgian Government type) and 58/65; and in the latter case metallographical tests of weldability were carried out. The tests for acceptance of electrodes comprised the following:

1) A tensile test on cylindrical specimens of 10 mm diameter consisting entirely of the deposited metal and serving to determine the breaking stress, the apparent elastic limit, the elongation as measured between gauge points 50 mm apart, and the reduction in area.

2) A test for resilience, carried out on a Mesnager specimen of the small type which was cut from the mass of deposited weld metal. Alternatively, tests were carried out on specimens connected by a V weld in which the notch was placed

either at the top or the bottom of the V, or along the bisecting line. The parent metal was steel 42/50 or stèel 58/65. These tests, which were less regular, support a recommendation of that form of specimen in which the notch is cut in an adequate volume of weld metal, at a sufficient distance from the parent metal. The specimens consisting of a simple V weld may be suitable as a tests of weldability, but it is necessary to define carefully the position of the notch, in relation to the very limited volume of the weld metal.

3) A bending test carried out on a steel plate of 42/50 steel 10 mm thick after working, 200 mm long, and 40 to 70 mm (averaged 50 mm) wide, containing a V weld which was required to be bent over a mandril of 30 mm diameter until the two ends were parallel (180°), the weld being exactly on the axis of the bend and the point of the V being in contact with the mandril. (This last test might be omitted or might be used solely for the qualifying of welders.)

The tests used for approving welders (already trained and qualified) included the following:

1) A bending test as described above, and in cases where steel 58/65 was to be used a similar test carried out on a mandril of 75 mm instead of 30 mm diameter. The specimens may be welded either horizontally or vertically according to the nature of the jobs to be carried out.

2) A somewhat special kind of bending test made on a cruciform specimen similar to that laid down in the German regulations. The cross has two branches of 150 mm total length, 100 mm wide and 15 mm thick. One branch consists of two pieces welded to one another by angle welds, either single or K. This cross is subsequently flattened in a press along one of its diagonals, first until the two branches are parallel and are 30 mm apart, and then 15 mm only. The parent metal was steel 42/50. This test is relatively severe, especially in the case of V or K welds. The welds on these crosses were carried out horizontally or vertically according to the conditions.

After qualification the welders were periodically subjected to control tests, which consisted of ordinary bending specimens in according with 1) above. It is desirable, with all bending specimens, especially those used for the qualification of welders, to add a metallographic test or a series of hardness tests with the Brinell or Rockwell hardness measuring instrument, to examine after sawing in two, or to use X-rays. The hardness test is useful as a means of checking the quality of the parent metal and of detecting any possible heat treatment that may have been applied to the specimens, while the sawing or X-ray test serves to reveal the degree of regularity and the detailed quality of the weld.

The welders having thus been checked, an organisation was established for identifying any given weld in the job by reference to a register in which all the welds carried out by the different welders were accurately recorded together with any relevant observations. Control over the intensity of the welding current by reference to ammeters, which is practised from time to time, may be generalised as considered necessary. Though these methods for the acceptance of materials, the qualification of welders and the supervision of the work, afford no absolute guarantee, there can be no doubt that they are far in advance of those practised for the control of reinforced concrete work. Concrete workers have for long not been subjected to any qualifying tests, and even the vibration of concrete is not made subject to personal guarantees comparable to thôse enforced in the welding of bridges and frames.

Many forms of control are possible after the welds have been carried out, the simplest being to check the dimensions of the angle fillets by means of gauges in convenient sets. In the case of V and X welds the actual shaping of the pieces to be joined, which is checked before welding, serves to determine the dimensions of the welds. It is necessary, also, to check whether any relative displacement or deformations of the pieces to be connected has occurred.

There follows an examination of the appearance of the weld, and this may be rather misleading unless the peculiarities of the welder are known. Certain features require special notice, such as the craters, the beginnings of the runs and the cut made into the parent metal. Acoustic testing with a hammer, even using a stethoscope, is not an effective method, except in the case of a serious defect which would be visible to the naked eye.

Non-destructive methods of inspection such as by magnetoscopic and radiographic apparatus, etc., often seem inconvenient and unsuitable for general use on the site or even in the fabricating shop, but this application so far as possible is certainly to be desired. The paper by M. *Berthold* opens up some interesting ideas, but it implies an organisation which would not suit the conditions in all countries, and its general adoption is problematical. Magnetoscopic examination would appear to be fallacious. The method suggested by *Schmuckler*, making use of check borings, is practical enough but of limited scope. It has been applied to the structures mentioned above, but as these consisted of steel 58/65 parent metal with hard welds it was difficult to bore the weld beads and the operations were somewhat slow and costly. Altogether 73 such tests were carried out in 595 tons of steelwork (1 test for 8 tons). Out of 73 tests 5 showed important defects such as holes of notable size at the bottom of the angle fillets, and 9 showed slight defects such as small air bubbles. A few cracks in transverse welds connecting plates to the flanges of beams were noticed in the shops these welds having been made in very cold weather, and also in a few unimportant welds at the ends of the rail-bearing beams.

This control, which was made pretty extensive both in the shops and on the site — not a usual practice in Belgium — showed that notwithstanding the precautions taken to guarantee the quality of materials and the labour, the welds contained a moderate percentage of imperfections. This conclusion justifies the opinion put forward that it is necessary to take account of contingencies of this kind when designing welded structures if adequate safety is to be ensured. As stated at the beginning, the parent metal may have as many defects as weld: a fact disclosed by the *Baumann* results and by macrographical and micrographical examinations and also by such occurrences in welded structures as the doubled [or foliated] plates, internal stresses, local cold working effects, beginning of cracks, over heating, etc.

It is no bad thing that the designer should have impressed upon him the idea that the materials he has to use in his work are not perfect. Such an idea is preferable to fallacious belief in a perfection which cannot be realised, and serves also to moderate the reliance placed on that delicate and ambiguous phenomenon known as adaptation. It requires, for the design and execution of welded work, technicians of high education and high personal and professional qualities. Moreover, the strictness of a control which may be absolute but is necessarily *a posteriori* must be qualified, in the practical construction of bridges and frames, by the admission of a certain tolerance, or reasonable regard for the interests to be served. This is the upshot of M. *Berthold*'s remarks on the subject of X-ray testing. For safe and economical construction — which is the engineer's ideal — the aim should be to exercise control over welding by the use of methods which are adequate, without being excessive. The most useful form of control will undoubtedly be the behaviour of the structures in service, especially in the case of bridges, and this may be checked by periodical inspections of the welds using whatever means are preferred, analogously to the periodical inspection of rivets.

(Five slides which are not reproduced here were shown at the meeting of the Congress.)

IIIc 3

Quality Control in Welding.

Prüfung der Güte der Schweißungen.

Contrôle de la qualité des soudures.

A. Goelzer,
Directeur de la Société Secrom, Paris.

To obtain good results in welded structural work, control over the quality of the welds is essential, for the situation is one in which a relatively new method of forming connections has to defend itself against all possible risk of failure. The specifications and regulations which govern welding lay down certain tests which are designed to control the quality of welds as conveniently as possible. Since the quality of a weld depends firstly on the intrinsic qualities of the weld metal and secondly on the skill of the operator, there arises a need for the following tests, as laid down in the relevant French regulations.

a) *Tests of weld metal.*

These tests are for tensile strength and for resilience. The specimens are taken in the first place from the same metal as is used for the electrodes and are cast in a steel mould. The tensile tests are required to give the following results:

	Parent metal	
	Ac 42	Ac 54
Minimum tensile strength	38 kg/mm²	48 kg/mm²
Minimum elongation at fracture measured between gauge points	15 %	12 %

At the same time the resilience must not be less than 8 kg/cm³.

b) *Tests on welded joints.*

These tests serve to control both the quality of the weld metal and the proper execution of the welded work. They include tensile tests and bendig tests to be carried out on specimens made by butt welding flat plates to one another.

The tensile tests must give a value of not less than 42 kg/mm² if the parent metal is "steel 42" and not less than 54 kg/mm² if the parent metal is "steel 54". The fracture obtained from the weld must show neither air bubbles, dark zones, slag incusions nor scoria.

The bending test is made over two cylindrical supports 100 mm in diameter placed at 150 mm centres, the weld being at an equal distance from either support and the opening of the V being downward. By means of clamps applied on the right of the weld a press is operated and until the two branches of the welded plates form an angle of 60°. There must then appear no flaw or crack on the tension side of either the weld or the parent metal.

These different methods of control may be combined with the examinations for appointment of welding workmen. From a practical point of view the best guarantee is not to employ any workmen without adequate training, and to test their skill by periodical examinations.

Apart from the more or less official point of view described above, various attempts have been made to perfect direct methods of inspection which will serve the purpose of identifying such defects as are liable to arise in the miniature metallurgical operations which appertain to welding. The chief of these methods are the following[1]:

Radiographic examination.

Radiography is applied to welds by means either of radium or of radon. It may be recalled that radium is transformed into radon by the emission of α rays which consist of atoms of helium carrying double positive charges in rapid motion. The radon in turn is transformed into radium B, and thence into radium C, by the emission of β and γ rays respectively. The β rays consist of electrons in very rapid motion. The high velocity α and β rays are physiologically dangerous and cannot be used for purposes of radiography; they can be screened off by the use of copper, silver or platinum bombs which allow the γ rays to pass through. By means of radiography it is possible, for instance, to photograph welds in hollow bodies by placing a suitable capsule inside the piece to be photographed.

Magnetographic examination.

The magnetographic method of examination, due to Professor *Roux* of the Ecole Centrale des Arts et Manufactures, is based on the following principle. If a sheet of metal covered by a sheet of paper is placed over a magnet and iron fillings are scattered over the paper the result is to form a magnetic spectrum, the character of which is well known. If, now, the single sheet of metal is replaced by two sheets properly welded together — that is to say without air bubbles or defects of any kind — the line of welding is revealed by the spectrum, because the magnetic permeability of the weld metal differs from that of the sheet in consequence of the greater thickness of the latter. If the weld has been well made the spectrum of the line of welding is regular and is free from any kind of anomaly, but each of the common faults of welding may be recognised by a characteristic figure; for instance, if there is a lack of penetration, which is a fairly frequent fault, a black line appears, due to the increase in density of the lines of force in the thinner portions. Again, if there

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[1] See «La soudure à l'arc électrique et la soudure à l'hydrogène atomique» by Dr. *Maurice Lebrun'* of the University of Paris.

is a complete absence of welding at the middle of the thickness intended to be welded, a more distinct black band appears.

The magnetographic method allows of welds bein examined in their actual positions provided that the piece to be examined is not too massive, but it is not possible, for instance, to examine in this way the hull of a large ship. In order to retain a record of such examinations use may be made of transparent paper covered with an adhesive solution on to which the filings are thrown. The method can be worked in any position, and portable apparatus has been developed for use in checking welded work on the site.

Magneto-acoustic examination.

The complement to the Roux method is the use of listening apparatus, and this nomises to yield results of considerable interest. The device consists in creating a magnetic field in a welded plate by means of an electro-magnet and inserting in this field a small coil to which a periodic motion is imparted. By this means a tension is induced in the coil, which is proportional to the variation in the magnetic field along the weld over which the coil is moved. Such induced tensions give rise to harmonic waves, and the latter are strengthened by an amplifier, similar to those used in wireless apparatus, and detected in a headphone. The disadvantage of the magneto-acoustic method is that the recognition of possible defects in the welds is made to depend on a personal factor.

Direct examination by boring.

This method consists in the use of a special form of milling tool to withdraw from the metal a small cylinder which may be subjected to macrographical examination. The Schmuckler tool has been specially devised for this kind of examination. The advantage of the method is that it gives a direct control which cannot be disputed; its disadvantage is that it can only be performed by drilling in depth.

Mention will now be made of two practical methods that can be applied to electric arc welding.

Control of the electrical characteristics of the arc.

A defect common to the methods explained above is that they afford a check only on *a posteriori*. It is possible, however, to control the welding while it is actually being performed, by reference to the characteristics of the arc and to the strength of current in amperes. (There is nothing to be gained by checking the difference in potential across the terminals of the arc.)

Without going so far as to use a recording ammeter, there are also portable apparatus, which do not involve any interference with the electric circuit but allow the current to be checked at any given moment. These instruments work equally well with direct and alternating currents.

If the current is correct in relation to the diameter of the electrode there is a certainty that all the metal which is deposited will be actually welded. There may still, of course, be discontinuities in the weld, but that is a defect which can easily be detected by an hydraulic test.

To adopt this method is to pass from checking the deposited metal to checking the workman who deposits the metal; a further step is to note the time it takes him to do so.

Control of welding time.

By means of a shunt an electric clock may be introduced into the welding circuit for the purpose of registering exactly how long the welder is at work. The time is measured in hundredths of an hour. The clock stops during the periods that the welder is not at work, and even during the periods that the electrode is short-circuited.

The control tests mentioned above relate only to the breaking strength against statical forces, and to measurements of resilience. For some time past a good deal of attention has also been given to fatigue tests on welded specimens. These tests are designed to throw light on the unfavourable effects from the point of view of resistance to fatigue that may be caused by welding.

The systematic researches of Mons. *Dutilleul,* a marine engineer, have shown that whenever a reduction in the fatigue strength has been detected in welds, by comparison with sound plate, the cause has nearly always been the existence of air bubbles in the welds, that is to say, porosity.

There is some tendency to look upon fatigue resistance as an absolute criterion. It would appear, however, that its chief importance is in relation to pieces which will actually be subject in service to alternating stresses repeated an indefinite number of times, as occurs in aeronautical and mechanical work; on the other hand, so far as contemporary structural engineering is concerned, the value of fatigue tests is open to a great deal of question. It may further be remarked, in this connection, that very often the fatigue strength and the resilience vary in opposite directions.

IIIc 4

Workshop Control of Welding.

Werkstattprüfung der Schweißung.

Le contrôle des soudures à l'atelier.

W. Heigh,
Welding Superintendent, Babcock & Wilcox, LTD., Glasgow.

Fundamentally, if the electrodes used have the essential characteristics, the control of the quality of welding depends on control of welders.

Procedures must be established for all welding conditions, and when those are tested and proved satisfactory the methods of making the weld should thereafter be a drill which the welder learns by heart.

It may be of interest to state that it is found that such a method not only obtains consistent welding but speeds up the actual operation. The reason is fairly apparent. When the welder knows exactly what to do he wastes no time in thinking out how it should be done.

The principal part of every procedure is the first run, whether the weld be a fillet weld or a butt weld. It takes a higher degree of skill to make the first run in any weld in any position (horizontal, vertical or overhead) than is necessary for subsequent runs. The usual faults in first runs are lack of penetration and cracking. Even cracking may be controlled to some extent by the skill of the welder.

It is usually found desirable to concentrate training of the welder on the elimination of slag pockets and lack-of-fusion lines. Procedures are chosen to suit.

The best methods of observing the degree of skill obtained is by taking a Radiograph of a butt weld or etching a number of sections from any kind of weld. Those are shown to the welder.

The value of those methods of showing a welder the faults in his work is much greater than that of all others, because they both give him a comprehensible picture. Sets of figures of mechanical tests are meaningless to the operator, at least at this stage. The only other picture of the inside of a weld which can be offered is obtained by breaking a weld and offering the break to the welder with the necessary explanations. The explanations usually confuse the simple facts and quite frequently the slag pockets are not revealed even to the trained observer. X-ray photographs and etching of sections are the most convincing methods of showing a welder his faults.

Given a close training in standard drills or procedures — by gradual steps from the horizontal to the overhead, and finally by composite drills for welding a butt weld and a fillet weld an a pipe of small diameter in a fixed position —

the mechanical results of welders tests is found to be invariably quite good. The only failures met with in a large concern using 130 welders have been with men who to be finally dispensed with as no suitable for employment as welders.

The principal deficiency in men who are incapable of being welders appears to be that they cannot see the weld they are making, or cannot see it intelligently perhaps a species of colour blindness in some cases and merely lack of intelligence in others.

Mechanical results in vertical and overhead position welds invariably passed the specifications for the class of work in which the electrodes and welders concerned were employed. Also, the only variation in the test results appear to depend entirely on the class of electrode used.

While the methods described are used to train men in making welds in vessels and pipes operating at pressures of upwards of 1000 lbs/sq. in. it is found that the degree of excellence acquired through time enables us to get good quality and fast welding on all kinds of work with the welders who have gone through the whole training.

IIIc 5

The Testing of Welds.

Über die Prüfung von Schweißnähten.

Le contrôle des soudures.

Dr. Ing. habil. A. Matting,
Professor an der Technischen Hochschule Hannover.

The importance of the personal element in welding makes it necessary that careful supervision should be supplemented by testing of the seams, and apart from this the welders themselves must be subjected to tests at regular intervals (see for instance DIN 4100). Such testing must be carried out rapidly and by simple means, and must yield conclusive results.

The bending test is very simple, and is carried out in Germany as indicated in Figs. 1—3. The scientific value of this test is a matter of dispute[1] since it is subject to very wide variations (as in the bending test, the quenching bending test, etc.) which have a considerable effect on the values obtained. In spite of much criticism the test is widely applied, especially in workshops. For high quality welds it has not been found to constitute a sufficient criterion.[2]

The tensile test is principally of importance in laboratory work. Various types of test specimen are adopted, the usual one for butt welds being that shown in Fig. 4. In the round notch bar breakage is forced to take place within the weld seam, and if the protrusion is smoothed down this serves as a test of the material. The prismatic form of bar which allows breakage to take place also in the transition zone or in the material itself, is intended as a test of workmanship. The determination of yield point and elongation is difficult and uncertain.

In structural steelwork great use is made of the cruciform test specimen which serves for testing fillet seams (Fig. 5). The requirements are closely defined in DIN 4100.

The significance of fatigue bending and fatigue tensile tests is being more and more recognised. With proper design, seams free from defects, and a gradual transition between the parent metal and the weld metal,[3] values of 15 kg/cm^2 are being obtained and even exceeded. Properly formed welded connections are as good as or even better than rivetted connections.[4] No standardised dimensions of specimen for fatigue tests have as yet been established.

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[1] *G. Fiek* and *A. Matting:* Autogene Metallbearbeitung 27 (1934), N° 4, p. 61.

[2] *A. Matting* and *H. Otte:* Ibid 29 (1936), N° 19, p. 289.

[3] *A. Matting* and *G. Oldenburg:* Elektroschweißung 7 (1936), N° 6, p. 108.

[4] *O. Kommerell:* Erläuterungen zu den Vorschriften für geschweißte Stahlbauten. II. Vollwandige Eisenbahnbrücken. Wilhelm Ernst & Sohn, Berlin 1936.

Hardness tests serve, in the first place, for the examination of deposition welding. The notched bar impact test (Fig. 6) is carried out in the case of structural steelwork under DIN 1913 only for the purpose of testing electrodes in the case of heavily loaded welded connections. This test is preferred as an acceptance test for the use of welding rods, the value required being between 5 and 7 kgm/cm², and as a rule this is obtained without any difficulty.[5]

For the assessment of welding rods, apart from the mechanical and technological method of testing, the bead test (Fig. 7) is also used, in order to indicate whether such rods can be used also for welding in difficult positions. Welding rods for gas welding and also bare electrodes as a rule give a good bead, but this characteristic may be impaired as the carbon content increases, and it is more difficult to obtain good beads with covered electrodes, especially if the covering is thick, though excellent results have sometimes been obtained. In testing the adhesion of covered electrodes in accordance with DIN 1913 use is now made of vertical fillet seams, one half of the length being deposited upwards and one half downwards. The bead allows conclusions to be drawn as to the performance of the welding rod in overhead welding at the same time.

Specimens composed entirely of weld metal have not hitherto been much used. The determination of deformability in welded specimens is difficult and unreliable. A proposal to carry out measurements of elongation on cruciform specimens[6] is now being investigated. In the stretching test,[7] Fig. 8, a proportional bar with a longitudinal weld seam is used, the proportion of the total cross section occupied by the weld being about 30 %. The specimen is stretched in a tensile testing machine until the capacity of the weld for elongation is used up. The difference in elongation between different kinds of welding rod, the effect exerted by the nature of the material and the effect due to the welding method may in this way be readily estimated. Specimens without a reinforcement as a rule give from 2 to 3 % higher values of elongation. This form of specimen has not yet come into general use.

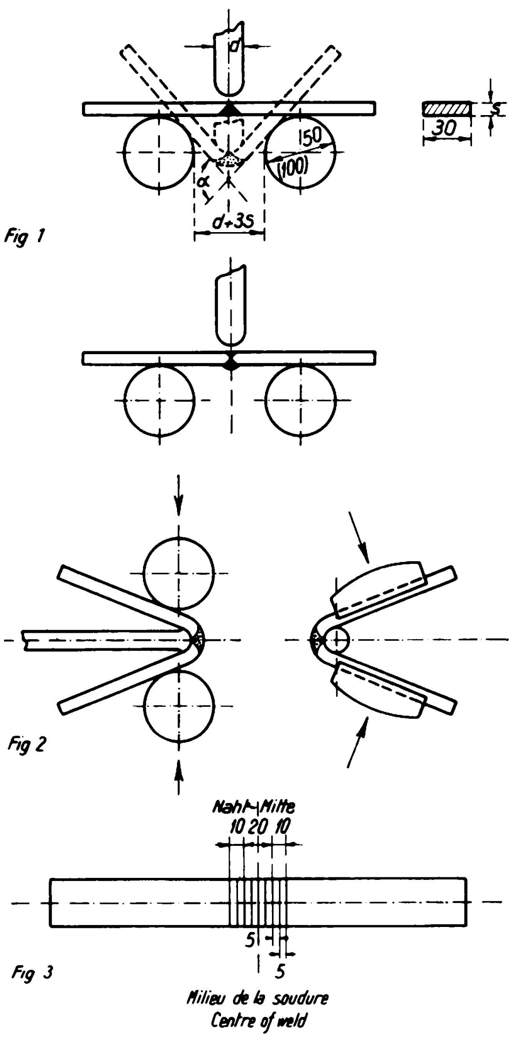
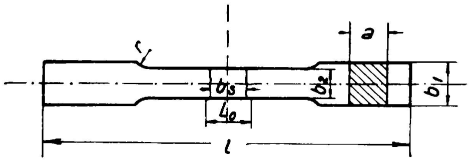


Fig. 1—3.
Arrangement of bending test in accordance with the provisional DIN standard. DVM A 121.

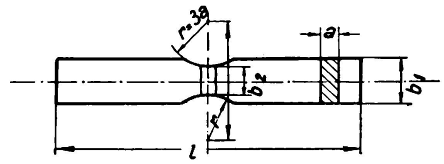
[5] *P. Bardtke* and *Matting:* Autogene Metallbearbeitung 26 (1933), N°18, p. 279 and N°19, p. 290.
[6] *H. Blomberg:* Elektroschweißung 6 (1935), N° 4, p. 61.
[7] *A. Matting:* Elektroschweißung 7 (1936), N° 3, p. 53.

These are the methods of testing which enable both welders and forms of welding connections to be easily supervised, and examples of these are provided by the wedge and angle tests shown in Fig. 9. More accurate indications are not required in these cases. Frequently, also, specimens are cut out from the current work in hand, and are subjected to suitable destructive tests.



Thickness a	10[1] to 25	over 25 to 35	over 35 to 45
Length of bar l	250	300	350
Gauge length Lo	should equal the width of weld $b_s + 5$ to 10 mm		
b_1	30	35	40
b_2	20	25	30
r	15	20	25

[1] For a = 6 mm the DVL test bar is to be used.



Thickness a	6	8	10	12	14	16	18	20
Length of bar l	250	250	250	250	250	250	250	250
b_1	18	24	30	36	42	48	54	60
b_2	12	16	20	24	28	32	36	40
r	18	24	30	36	42	48	54	60

Fig. 4.

Shapes of tensile test bars according to the provisional DIN standard. DVM A 120.

No numerical relationships can, of course, be obtained between the various methods of testing welded connections, except those between strength, elongation and hardness in the case of carbon steels. In the case of welds the conversion figure, that is to say the ratio of the breaking stress to the hardness number, is not 0.36 but is between 0.29 and 0.32.[8] The notched bar tenacity depends only on the structure and cannot be directly related either to the elongation at breakage or to the fatigue strength. There is also no satisfactory relationship between fatigue strength and tensile strength, yield points and elongation. There is, therefore, no way of avoiding the necessity for separate experiments to determine each of the properties it is desired to ascertain.

Examinations of the coarse structure, as illustrated in Fig. 10, are very suitable as a method of testing penetration and porosity, and for detecting slag inclusions. Microphotographs, as in Fig. 11, serve for amplifying such information and for detecting foreign matter. Special attention is now

[8] *A. Matting* and *H. Koch:* Elektroschweißung 5 (1934), N° 7, p. 127.

being paid to the behaviour of welded connections as regards corrosive influences.[9]

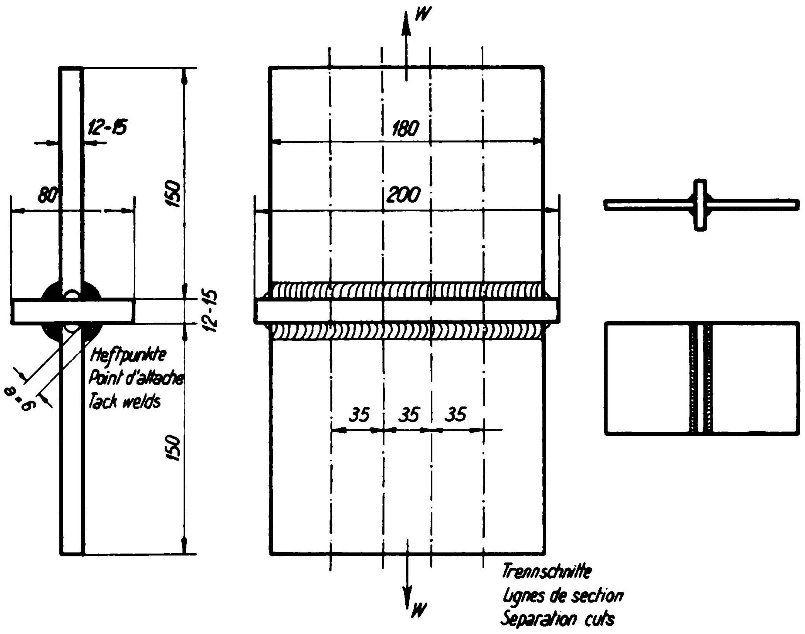


Fig. 5.

Testing of end fillet welds.

In order that, apart from considerations of safety, economic advantage may be realised from additional care taken in testing the welds, great importance

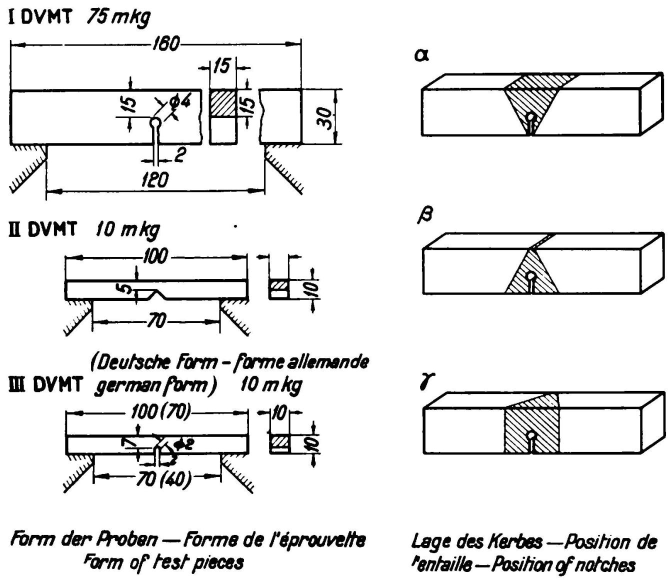


Fig. 6.

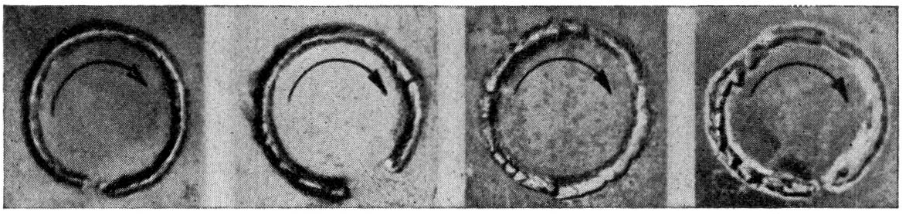
Notched impact bar specimens.

attaches to intelligent application and improvement of the testing methods.[10] The high demands which are now imposed upon weld seams have been rendered

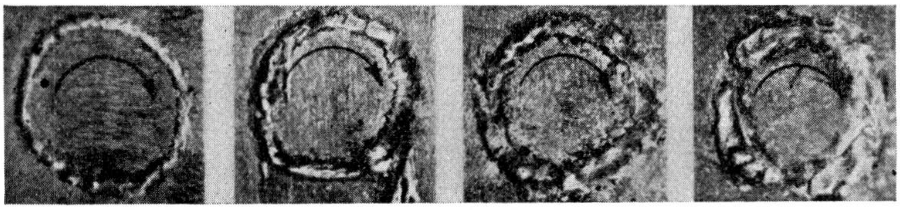
[9] *E. Diepschlag:* Autogene Metallbearbeitung 29 (1936), N° 8, p. 113.

[10] *H. Koch:* Stahlbau 9 (1936), N° 26, p. 206.

acceptable (apart from the improvement in human factor) only by the fact that it is possible to obtain seams of perfect quality.



90°	90°	90°	90°
bare	bare	covered	covered
— 4 mm dia.	— 5 mm dia.	— 4 mm dia.	— 4 mm dia.
			Overhead electrode.



45°	30°	15°	15°
covered	covered	covered	covered
— 4 mm dia.	— 6 mm dia.	— 8 mm dia.	— 12 mm dia.

Fig. 7.

Adhesion tests.

In the testing of finished work non-destructive methods are to be preferred to destructive. The act of weakening the seam by opening it up may indeed have an educational value, but apart from this it should be avoided except as a very rough and ready form of test.[11] The part affected can be rewelded, but in doing

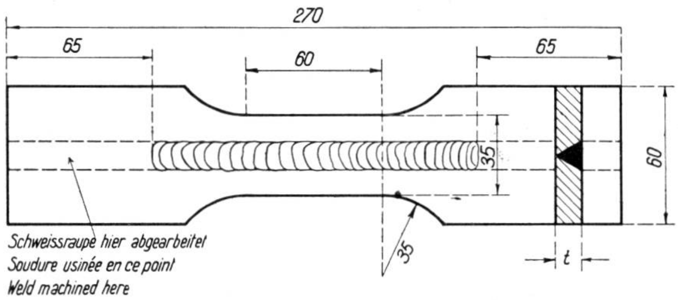


Fig. 8.

Tensile test bar.

this additional thermal stresses may arise, and moreover an unknown factor is introduced in place of the known (Fig. 12).

If the weld seam is not to suffer damage some non-destructive method of testing has to be adopted, and moreover this should be one which allows con-

[11] *R. Bernhard* and *A. Matting:* Stahlbau 5 (1932), N° 15, p. 114.

clusions to be drawn as to the quality of the seam. In the construction of containers, tests by water, air or steam pressure are possible, and in special cases

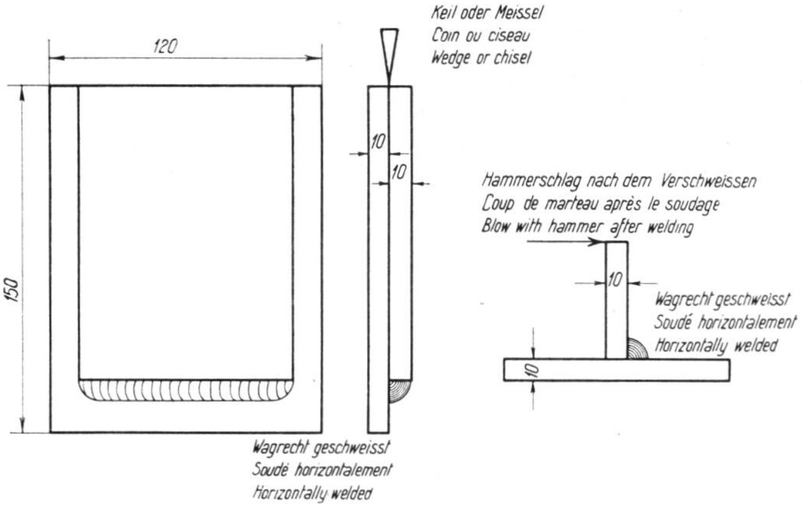


Fig. 9.
Wedge and angle tests.

tests may be carried out by means of explosion within the containers, though these are of course destructive in their nature.[12] In welded structures the place of these

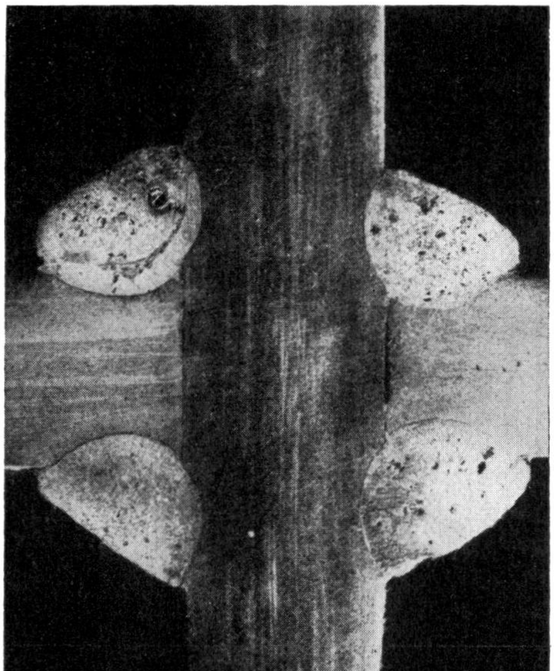


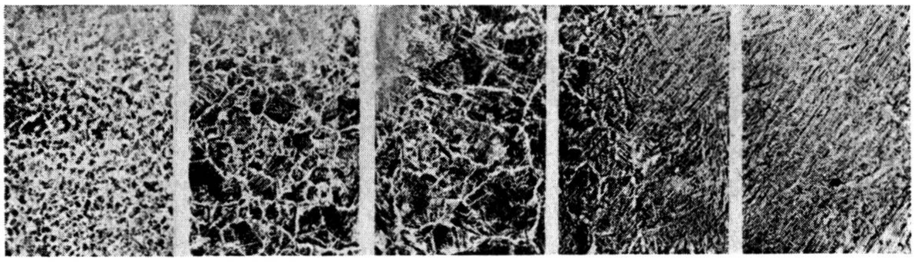
Fig. 10.
Coarse texture of fillet weld made with bare electrodes. Good penetration.

is taken by loading tests, or by fatigue tests combined with measurements of stress.[13]

[12] *E. C. Hutchinson:* Power, 7th Oct. 1930.

[13] *W. Rosteck:* Organ für die Fortschritte des Eisenbahnwesens 1934, Nos. 10 and 11, pp. 187 and 197.

Attempts to examine weld seams accoustically or by reference to electrical fields of stress have been without success, but magnetic methods have been more suc-



Gas fusion weld.

Parent metal Transition Weld

Electric arc weld.

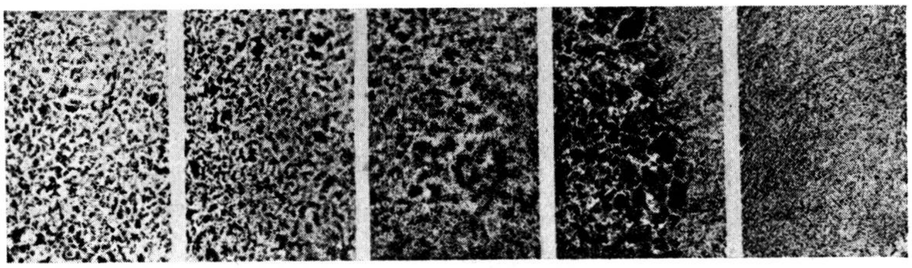
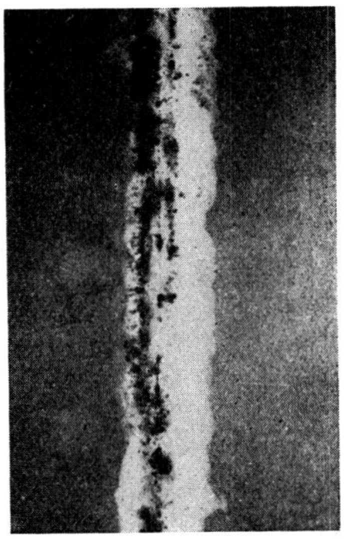


Fig. 11.
Fine texture of gas and arc welds.

cessful. In these the work is magnitised and iron filings are scattered upon it, the uniform arrangement of which would be disturbed if there are any hollow places, slag inclusions or defects of bond.

Elevation



X-ray negative.

Fig. 12.
Bad arc weld.

In the author's opinion, however, the electro-magnetic acoustic method of testing welds has not fulfilled expectations.[14] The weld seams are here explored electromagnetically and the impulses of current are rendered audible in headphones. It is not, however possible to locate defects definitely by this means.

Much the best method of testing is by radiation, particularly by means of X-rays.[15] Gamma rays can also be used for testing,[16] but in structural steelwork this method does not come into question. In the examination of the coarse

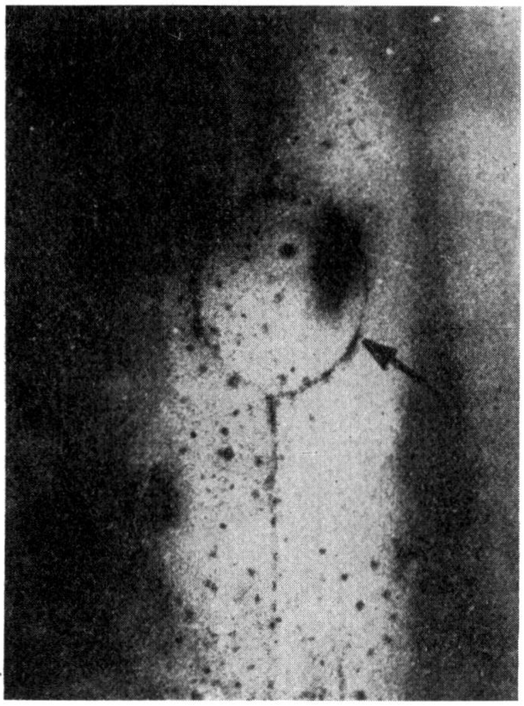


Fig. 13.

X-ray negative showing faulty welding of a hole.

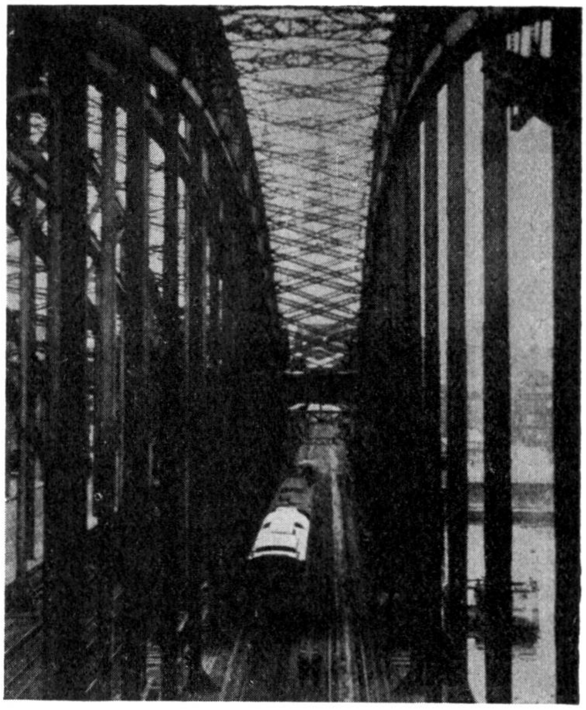


Fig. 14.

X-ray testing car on the Hohenzollern Bridge in Cologne.

structure by X-rays the image may be thrown directly on a screen or may be rendered visible on X-ray films provided the thickness of the work is not excessive (Fig. 13). Apparatus has been so developed that such tests may be carried out on the site and in actual service. Fig. 14 shows a portable X-ray testing set used for particularly difficult investigations. Figs. 15 and 16 show that X-ray tests may also be made on railway bridges. The limitations of X-ray technique lie in difficulties as regards apparatus, lack of sensitivity to faults, and the thickness of the material.

Non-destructive methods of testing may also be combined with destructive methods. It is a matter of dispute how far the results of non-destructive testing

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14 *S. Kießkalt:* Autogene Metallbearbeitung 27 (1934), N° 5, p. 65.

15 *A. Matting:* Anwendung der Durchstrahlungsverfahren in der Technik. Akademische Verlagsanstalt m. b. H., Leipzig 1935, p. 51.

16 *R. Berthold:* Z.V.D.I. 78 (1934), N° 6, p. 173.

can be linked up with those of direct testing.[17] By combining different methods of testing it is usually possible to obtain sufficiently conclusive results as to the structure of a weld seam.

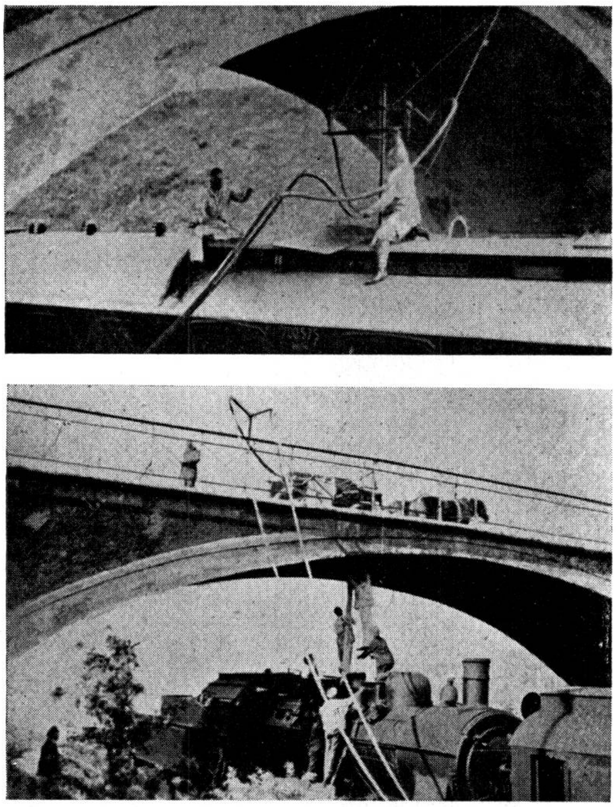

Fig. 15.
X-ray examination of a reinforced concrete bridge.

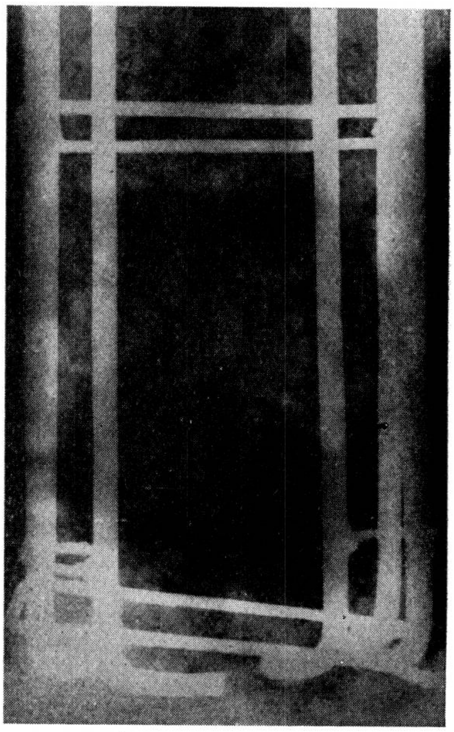

Fig. 16.
X-ray negatives of a reinforced concrete girder.

[17] *A. Matting* and *C. Stieler:* Stahlbau 6 (1933), N° 24, p. 185.

IIIc 6

Examination of weld-seams.

Prüfung der Schweißnähte.

Essai et contrôle des cordons de soudure.

Dr. Ing. h. c. M. Roš,

Professeur à l'Ecole Polytechnique Fédérale et Président de la Direction du Laboratoire Fédéral d'Essai des Matériaux et Institut de Recherches pour l'Industrie, le Génie Civil et les Arts et Métiers, Zurich.

The examination includes:

1) Weld-rods (Electrodes),
2) Welders,
3) Welding-seams in the finished structure.

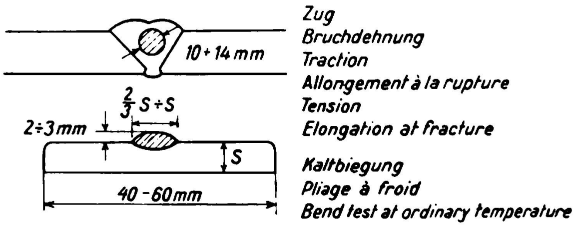

Fig. 1.

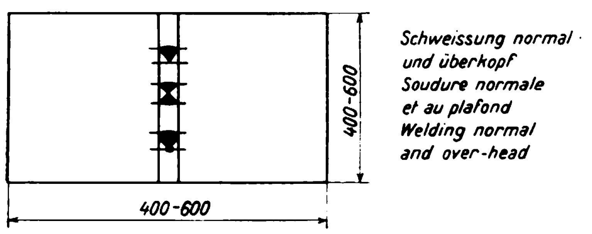

Fig. 2.

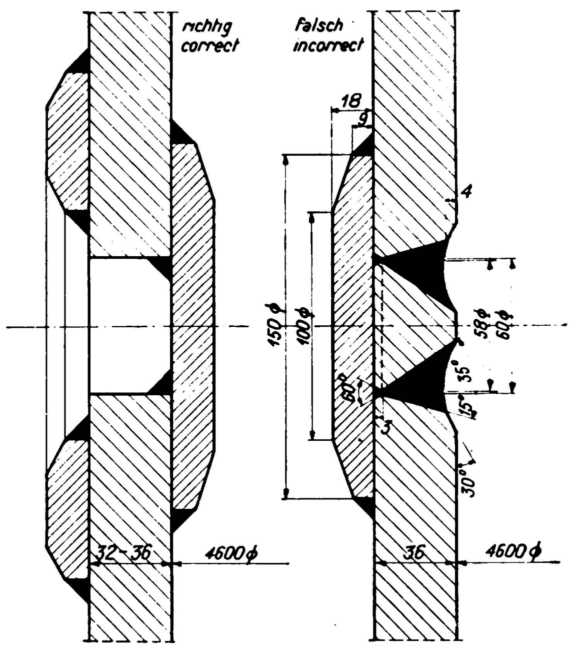

Fig. 3.

Correct and incorrect closure of place whence specimen has been removed, by welding.

1) *Weld-rods (Electrodes).* The melted weld-material is tested as regards strength, deformation-properties and sensibility with respect to quenching.

Test-piece taken from weld: — Weld material —

Required values:

Brinell-hardness H = 115 to 160

Tensile-strength for steel as normally used in construction (β_z = 36 — 44 kg/mm², C $\leqq$ 0,15 %): β_z = 40 — 55 kg/mm², elongation after rupture λ_{10} = 15 — 25 %.

Welding-"cords" laid down in thin layers — Sensibility with respect to quenching —

Flexure numeral: $K = 50 \cdot \frac{s}{r} = 32 — 48$.

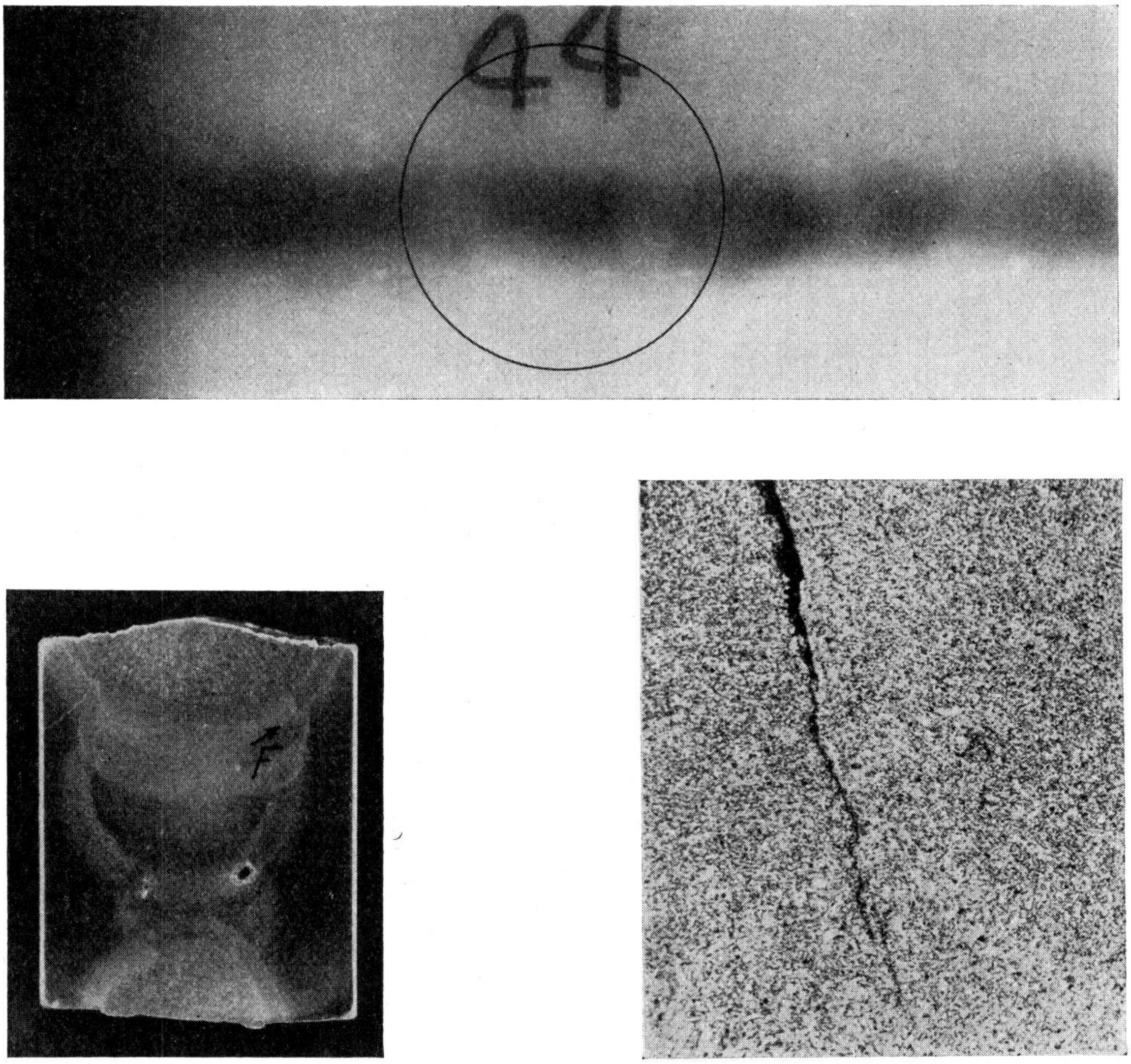

Fig. 4.
Microscopic crack in structure of weld metal, not detectable by x-rays.

Not compelling values:

Yield-point δ_s = 25 — 35 kg/mm²

notch-tenacity of small normalised test-pieces EMPA x $\geqq$ 4 mkg/cm².

For high-grade steels separate agreements.

2) *Welder. — Workshop.* Plates and bars, of very small and large thickness as used in construction, welded in form of butt- and cruciform joints in normal as well as overhead position were examined by means of X-rays and afterwards subjected to the following tests: coarse-(macro) and fine-(micro)-structure, hardness, tensile-strength, folding-flexure capacity, repeated stress-strength and — by way of exception — to notch — tenacity. The selection of the test-pieces is carried out according to the results of the X-ray examination.

Failures of bond are not admissible. The structure must be free from cracks. For steel as normally used in construction the following is required: Hardness numbers of cross-section $H = 115 - 160$ kg/mm², surface $H \leqq 180$ kg/mm²; tensile-strength — butt-joint — equal to that of the steel, $\beta_z = 36 - 44$ kg/mm², tensile-strength — cruciform — joint average value $\beta_z = 25$ kg/mm², minimum 22,5 kg/mm²; folding-flexure capacity $K = 20 - 28$ (plate thickness $\delta < 12$ mm), $K = 16 - 20$ ($\delta = 12 - 20$ mm) and $K = 12 - 16$ ($\delta > 20$ mm);

Repeated stress-strength — blunt-joint -- $\sigma_U \geqq 15$ kg/mm² — normal position —
$\sigma_U \geqq 12$ kg/mm² — overhead —

Repeated stress-strength — cross-joint $\sigma_U \geqq 6$ kg/mm².

For high-grade and special-steels, requirements ad hoc.

3) *Welding-seams. — Finished structure.* Discs or bars of suitable shapes (round, oval), are be taken from the finished structure or structural element at suitable points and tested in a similar manner, to comply with the same requirements as regards strength and deformation, as described in the preceeding paragraph under „welders". The characteristic values obtained must be within the same limits; only very slight differences are admissible. The places from which the test-pieces are taken must be made good with particular care to avoid accumulations of weld-metal and to minimise internal and shrinkage stresses (Fig. 3).

X-ray examinations serve to disclose nonconnected spots, pores, slag inclusions and cracks, but they do not reveal the presence of very fine hair cracks which are often undesirable — Fig. 4. Frequently the X-ray examination must be carried out twice, firstly on the weld-seams after completion of the welding, secondly after removal (by shaping, grinding, cutting) of the uneveness of the top-layers. This applies also to places whence test pieces have been removed.

IIIc 7

Some Examples of Welded Steelwork in Czechoslovakia.

Einige Beispiele von geschweißten Stahlkonstruktionen in der Tschechoslowakei.

Quelques exemples de constructions soudées en Tchécoslovaquie.

A. Brebera,

Ingénieur, Conseiller Supérieur au Ministère des Travaux Publics à Prague.

The applications of electric welding in the special fields of bridge and large building construction show a great deal of progress during the last few years, due to the introduction of this process. Thus in 1935 a number of large hangars covering a total area of 1500 m² were constructed, the most notable part of the construction being the girder of 50 m span over an entrance (Fig. 1) which serves to carry lattice trusses spaced at 10 m centres (Fig. 2).

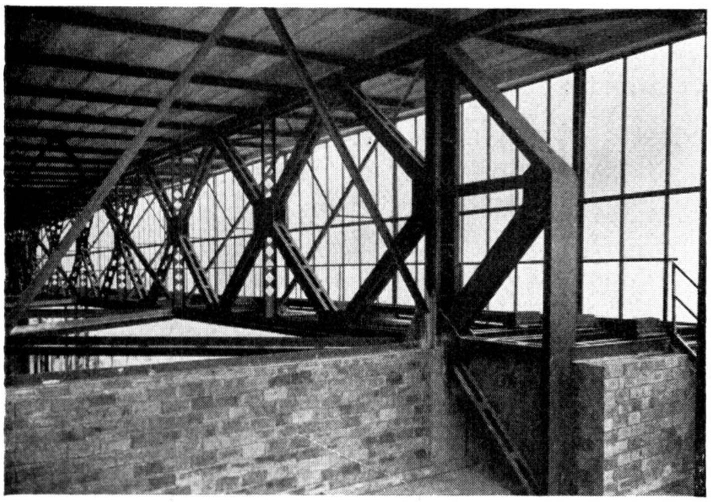

Fig. 1.

With a view to appraising the advantages of arc welding the whole design was worked out both for riveted and welded construction, and a comparison between the two solutions disclosed some interesting facts. At first the welded design for the 50 m span was based on the use of ordinary steel C 38, while the riveted design was made on the assumption that high tensile steel C 52 would be used; yet although in the second case the permissible stresses were taken 50 % higher than in the first, the weight of the girder worked out approximately the same in either.

As regards the actual supporting structure of the hangar for which ordinary steel C 38 was used in both cases, the saving in weight due to the adoption of welding worked out at 20 % (5,210 kg as against 6,500 kg). In view of these results, as well as economic considerations, the construction was, in fact, carried out in the ordinary steel C 38, electrically welded both in the fabricating shop and on the site.

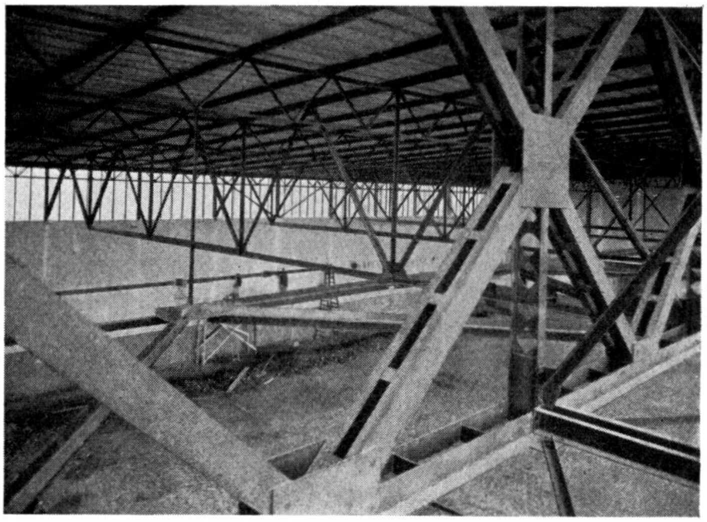

Fig. 2.

Two different types of electrodes were used in welding, giving the essentially different mechanical characteristics indicated by the minimum values specified as in Table I.

Table I.

Types of electrode	I	II
Tensile strength, in kg/mm^2	38	42
Elastic limit, in kg/mm^2	23	26
Elongation, %	12	20
Resilience (Mesnager), in kgm/cm^2	3	6

The use of Type I electrodes was authorised for various parts of the structure having a span of less than 15 m, as well as in side fillet seams of constructional members with a greater span, subject to a lower value for the permissible shear stress in these.

The maximum stresses allowed both in the parent steel and the weld metal are given in Table II in relation to the different stresses imposed.

Table II.

Permissible stresses	Parent metal	Weld metal	
		Type I	Type II
Tension	$\nu = 1200$ (1400) kg/cm^2	$0.75\ \nu$	$0.85\ \nu$
Compression	$\nu = 1200$ (1400) kg/cm^2	$0.95\ \nu$	$1.00\ \nu$
Shear	$\tau = 850$ (1000) kg/cm^2	$0.60\ \nu$	$0.65\ \nu$

Note. The values shown bracketed were allowed in cases where all external effects had been taken into account in the calculation, namely the effects of temperature and of wind pressure.

Before welding work was begun the types of weld seam and the welders were subjected to various tests, the specified minimum results of which are given in Tables III and IV.

Table III.

Tests for weld metal	Type of electrode I	II
Tensile strength, in kg/mm²	38	42
Shear strength, in kg/mm²	28	30
Bending angle, in degrees	120	180
Elongation, % . . . , ...	12	18

Table IV.

Tests for welders	Type of electrode I	II
Tensile strength, in kg/mm²	34	40
Shear strength, in kg/mm²	26	29
Bending angle, in degrees	90	120
Elongation, %	10	15

In the final design all the connections were worked out with a view to taking advantage of the latest improvements in welding technique. Considerable use was

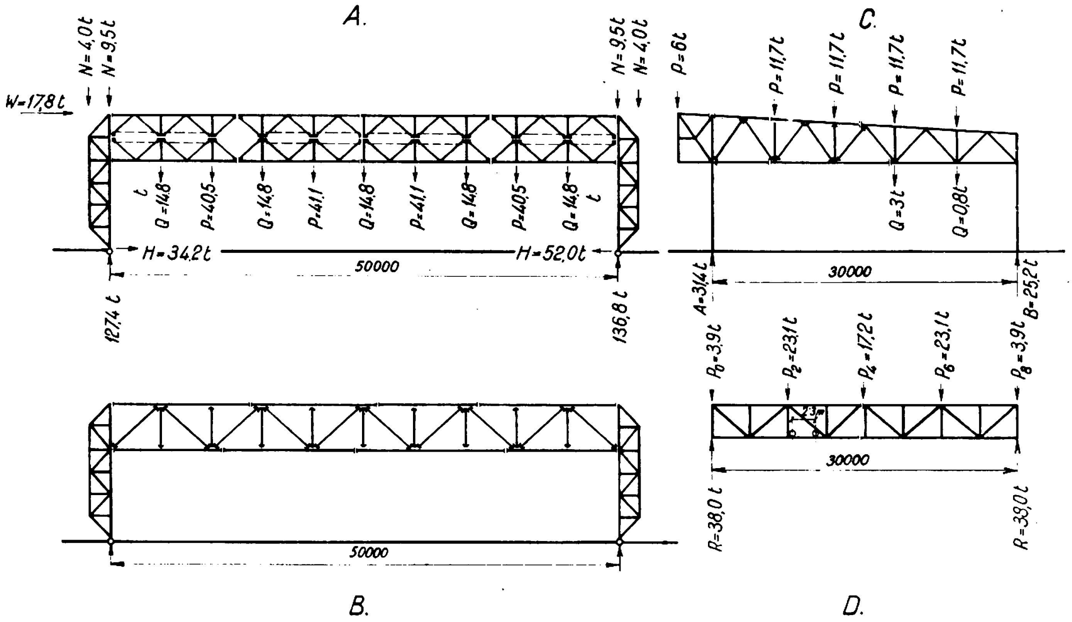

Fig. 3.

made of simple T sections obtained by cutting ordinary rolled joists in halves with the cutting blowpipe.

The boom member over the doorway was formed with a very simple cross section (500 × 500 mm plates and 100 × 180 × 18 mm angles) which was particularly well adapted to the conditions of stress arising therein, the total axial load in the boom being 318 tonnes.

Certain members such as the verticals of the girder over the doorway were formed of sections obtained by cutting ordinary rolled joists along a zig-zag line and then welding together the points of the two parts so separated after moving them opposite one another. In this inexpensive way it is possible to

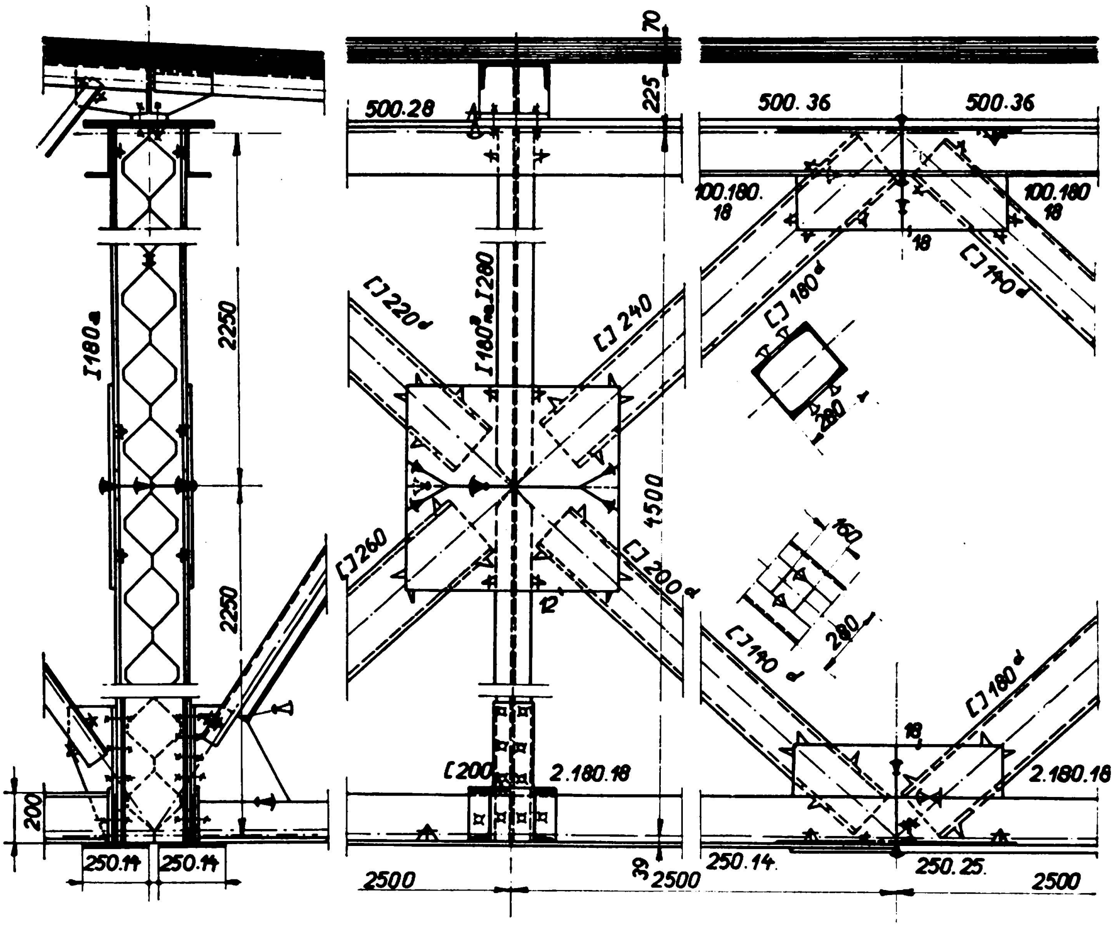

Fig. 4.

obtain an open webbed beam of the same weight as a normal beam but much more rigid.

For the most part, in connecting the various sections together, use was made of butt welds. The arrangement of the erection joints is shown in Fig. 3.

All the girders were assembled and welded on the ground, so far as possible in a horizontal position, before being offered up into their final positions and the last of the joints closed with them vertical. With a view to this procedure the erection welds were so arranged as to be easily accessible while being carried out.

The guiding idea in the design of the doorway was to reduce the number of site joints to a minimum. It was found possible to deliver the end verticals of the frame in single pieces, but the upper boom member (Fig. 4) was too long and too high for this to be done and had to be divided into a number of sections. In order to facilitate the assembly of these the lozenge type of

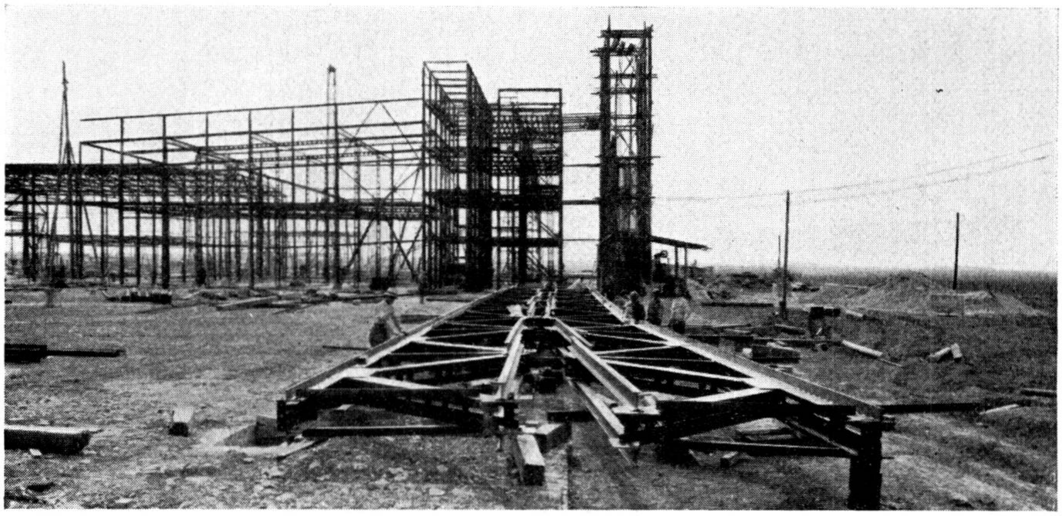

Fig. 5.

girder (Fig. 3) was adopted, thus reducing the number of sections to eight as against the 27 which would have been required with the usual triangular form (Fig. 3B).

With a view to rigidity combined with ease of transport, the various constituent parts were fitted to temporary boom members attached to central gussets, and, in an ingenious way, the roof trusses of the hangar were made to serve as booms (Fig. 5). The erection joists were placed in the boom members and in the central gussets as may be clearly seen in Fig. 3. The intersection of the diagonals is of some interest (Fig. 6). To facilitate the welding of the lower gusset the corresponding upper plate was notched, and its triangular complement was welded to the upper plate after the welding of the lower plate had been completed.

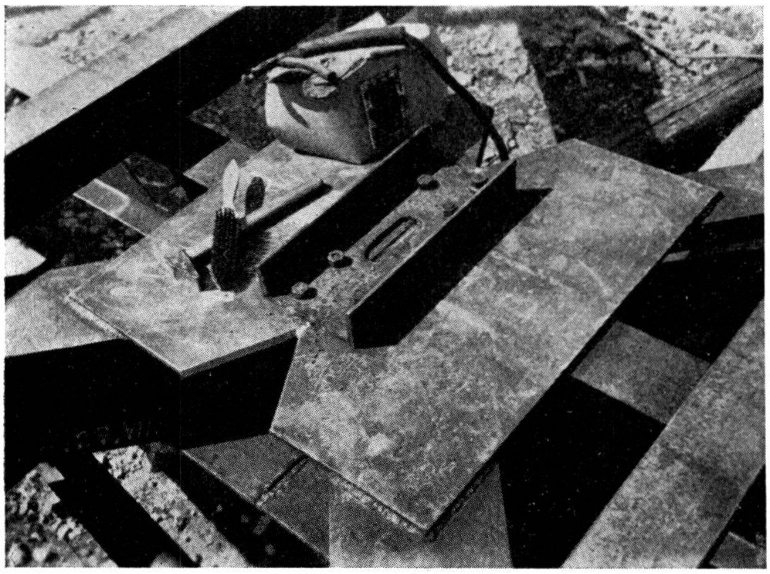

Fig. 6.

First the horizontal portions of the frame were assembled and clamped together with the aid of bolts in their correct positions; then the joints were tacked and completely welded in turn. Finally the auxiliary boom members were removed, and the footings were assembled and welded. The process of erection was begun by fitting the frames over the doorway, the whole frame, covering one span of 50 m and weighing 41.0 tonnes, being placed in position with the aid of erecting towers (Fig. 7), an operation which took four hours. The erection of the framework was then carried out in the usual way. Both in the shop and on the site the welding was done by means of direct current welding sets.

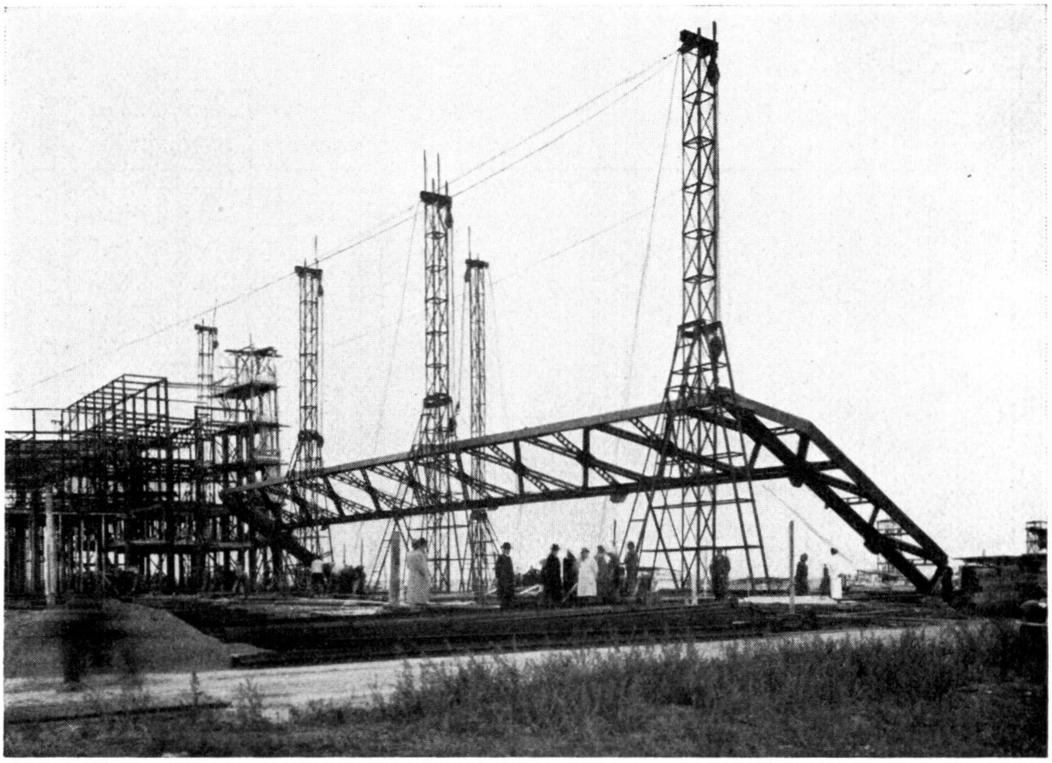

Fig. 7.

The construction of the steel frames was divided between the following two firms: —

S.A. des Anciens Etablissements Škoda, Pilsen.

Českomaravská-Kolben-Daněk, Prague.

The first mentioned firm made use of Böhler-B-Elite-KVA electrodes for welds of Type I, and of Arcos Stabilend electrodes for welds of Type II. The second firm made use of Elarc-Resistenz electrodes exclusively.

The average results obtained from the test specimens for electrodes, welds and welders are given in Table V, and against these results the prescribed minimum values have been included for comparison. Altogether 42 welders were tested in this way. It appears from the table that the minimum values, despite the high standard set, were easily obtained. The welding was closely supervised during the work, and on completion the welds were subjected to careful check and their dimensions accurately measured. A number of them were examined internally after drilling.

The design for the framework was carried out by the S.A. des Anciens Etablissements Škoda, Pilsen, with special attention to simplicity of execution both in the shops and on the job. The work was carried out under the control of the Bridge Department of the Ministry of Public Works.

Table V.

Acceptance test			Types of Electrodes: I		II		
			Böhler B-Elite	Minimum required	Arcos stabil.	Elarc Resist.	Minimum required
for electrodes	elastic limit, in kg/mm^2		30.9	23	35.0	40.0	26
	tensile strength, in kg/mm^2		46.5	38	46.3	48.7	42
	elongation, %		21.6	12	24.9	23.6	20
	resilience, in kgm/mm^2		4.3	3	8.5	9.7	6
for welds	tensile strength, in kg/mm^2		44.7	38	48.5	46.3	42
	shear strength, in kg/mm^2		34.1	28	34.6	37.1	30
for welders	tensile strength in kg/mm^2	horizontal posn.	47.2	—	49.6	46.8	—
		vertical „	42.2	—	47.9	48.0	—
		overhead „	43.8	—	50.5	47.0	—
		average „	44.5	34	49.5	47.3	40
	shear strength in kg/mm^2	horizontal „	33.3	—	33.3	35.7	—
		vertical „	33.7	—	35.7	36.1	—
		overhead „	31.3	—	35.1	34.2	—
		average „	33.1	26	34.8	35.3	29
for welds	I-shaped: tensile strength, in kg/mm^2		—	—	46.6	47.6	42
	V-shaped: tensile strength, in kg/mm^2		—	—	58.9	42.4	42

Another very large job was the welded construction of a road bridge of 52.005 m span (Fig. 8). Here the main girders were of the Vierendeel type without diagonals, this design being chosen mainly on aesthetic grounds but also because of the advantages it offers from the point of view of welding, and for the simplicity and rigidity of the intersections. Moreover in such a girder the secondary stresses are nil, whereas in a triangulated system they may vary between 10 to 15 % of the principal stresses, owing to the large sizes of gussets necessary at the intersections and to the system of calculation which has to be used.

Hence, using the same permissible stresses in the calculations, the true factor of safety is greater in the case of the Vierendeel girder, and finally bridges with Vierendeel main girders deflect much less than those with triangulated main girders on account of the great rigidity possessed by the intersections — a fact which is very important from the point of view of maintenance.

Hitherto the sole disadvantage attending the use of Vierendeel girders has been the difficulty of the statical calculations involved, but the Beggs-Blazek method of determining the influence line has completely removed this difficulty.[1] The advantage of this method lies in the fact that it is no longer necessary to rely on simplified assumptions and that the additional rigidity due to the fixation of the vertical members is automatically taken into account. The influence line can be accurately determined at any required point, and this makes it easy to check the conditions of stability.

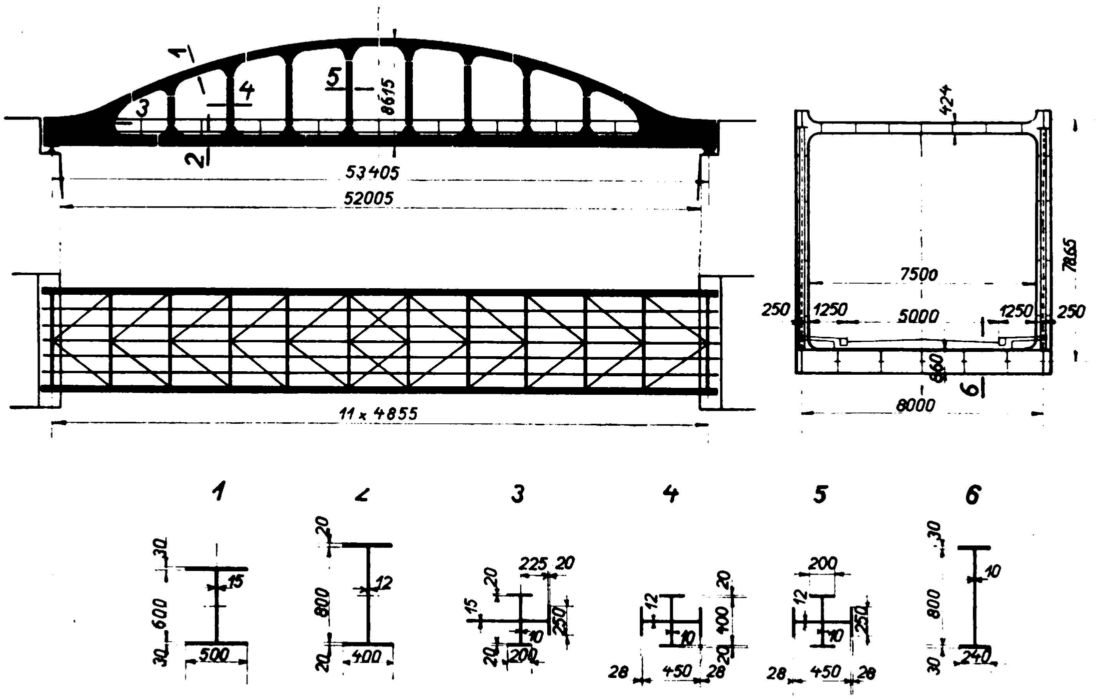

Fig. 8.

The girder is designed as hyperstatic to the 33^{rd} degree.

In addition the results obtained in this way were checked by reference to an approximate calculation in which it was assumed that the moments of inertia of the booms were constant in all panels and dependent on the length of the bars; at the same time it was assumed that no loads were applied except in line with the vertical members. These assumptions reduced the degree of hyperstaticity to 11 and rendered the calculations easier. The whole of this work was carried out in ordinary steel C 38 and entirely by the use of welding both in the shop and on the site. The welding was done exclusively with Arcos Stabilend electrodes.

[1] Final Report, 1^{st} Congress, I.A.B.S.E., p. 709.

The permissible stresses both for the parent steel and for the weld metal are indicated in Table VI.

Table VI.

Permissible stresses	Decking members		Main girder	
	Parent metal	Weld metal	Parent metal	Weld metal
Tension	$v = 850$ kg/cm²	0,75 v	870 + 3 l — maximum 1150 kg/cm² (1350 kg/cm²)	0.85 v
Compression		0,90 v		1.00 v
Shear	$\tau = 700$ (800) kg/cm²	0,50 v	700 (800) kg/cm²	0.60 v

Note. The values shown in brackets relate to the case where the calculations take account of all external forces (wind pressure).

For all the connecting welds the butt type has been preferred, and intersecting joints exposed to tensile stresses have been avoided on principle. Bracing members are connected to the verticals by means of butt welds. In order to avoid crowding the welds together the stiffeners of the bracings, booms and verticals have been holed at the angles, and this assists drainage.

The weight of the steel portion is 154 tonnes. The erection joints were arranged in such a way that the pieces could be delivered as large as possible (Fig. 8). The ends of the main girders, which are 9.293 m long and weigh 6.7 tonnes, were delivered to the site of the work in a single piece (Fig. 9).

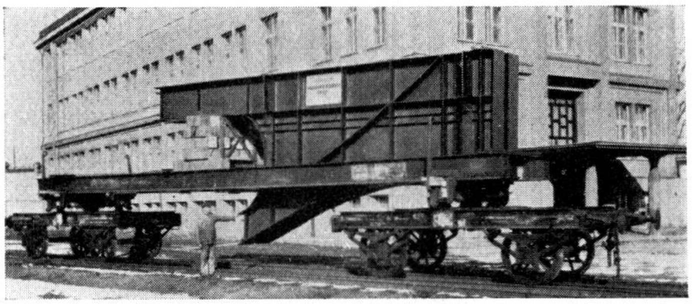

Fig. 9.

As the bridge was to be erected in spring time at the period of high water, the original intention was to carry out the welding immediately after the temporary erection by means of bolts had been completed, so that the welded points would be able, if necessary, to bear the dead weight of the structure in the event of the supporting falsework being damaged by the flood. The floor of the bridge was then to be welded in three sections, so as to lessen the stresses due to welding. The favourable weather encountered made it possible to modify these arrangements by welding the floor of the bridge to the lower boom members straight away, a procedure which helped to prevent the stresses due to the welding of the floor being transmitted to the main girders. Finally the verticals and other boom members were erected as soon as they had been welded in the shop (Figs. 10 to 12).

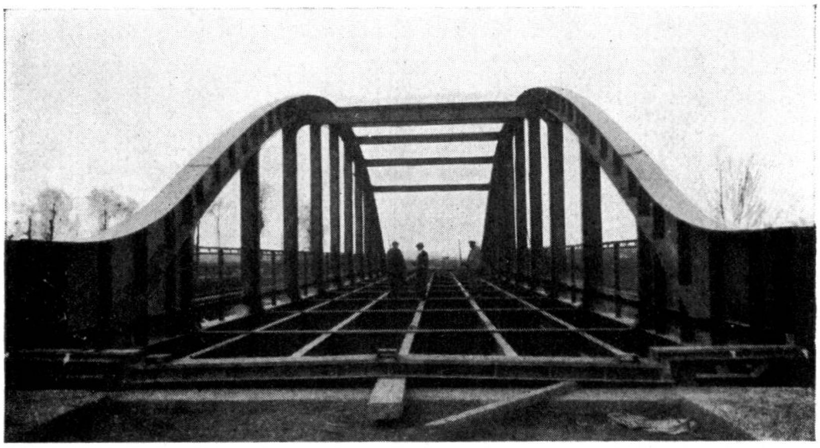

Fig. 10.

The main girders were given a maximum camber of 15 mm corresponding to the deflection under dead load together with half the life load.

In addition to the usual tests of steel, electrodes, welds and welders, fatigue tests were carried out. The fatigue limit for the weld was determined from *Wöhler*'s curve after carrying out eight tests to two million alternations of stress at 22 kg/mm² and also ten million alternations to 20.5 kg/mm². The tests were made on conical specimens in an Amsler fatigue testing machine.

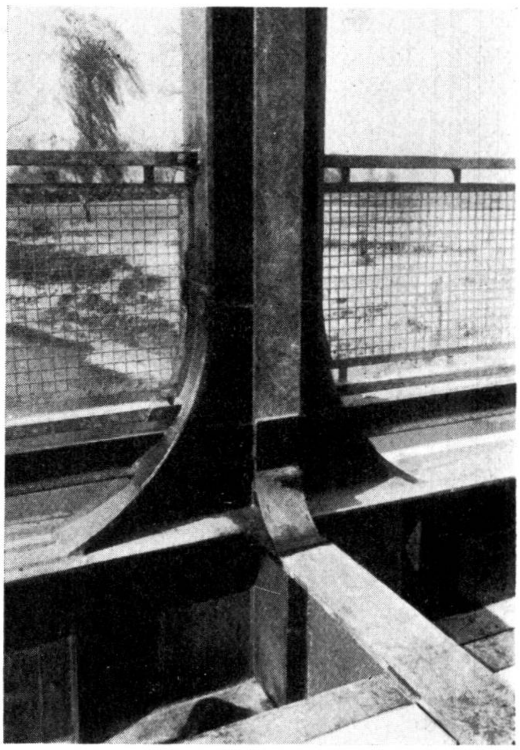

Fig. 11.

It further seemed advisable to carry out X-ray tests of the welds on a portion of the lower boom of the main girders (Fig. 13), and a model of the intersection of the lower boom was subjected to statical tests. These were carried out in the Laboratory for Testing Materials and Structures of the Czech College at Prague, enabling the stresses due to permanent loading and to assumed uniformly distributed live load to be calculated. Eventually it is intended to subject an intersection of this kind to fatigue test. When the whole structure is completed deflectometers will be applied to measure the deflections of the cross girders and main girders under stationary and moving loads.

The design of the bridge as a whole was carried out in the Bridge Department of the Ministry of Public Works, and the detailing of the final design as well as the construction of the work were entrusted to the S.A. des Anciens

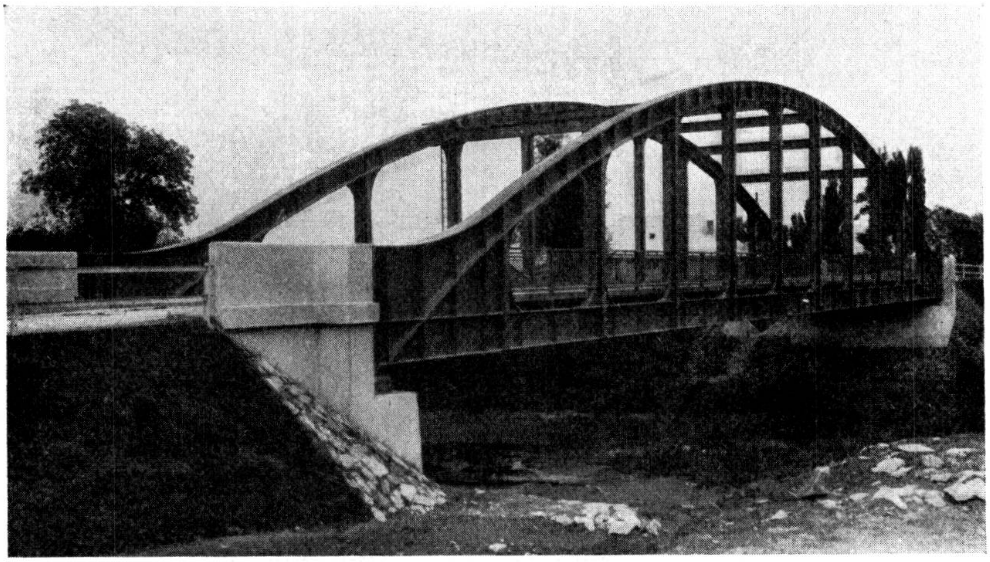

Fig. 12.

Etablissements Škoda, Pilsen, who completed the task to the entire satisfaction of the Ministry. The Bridge Department of the Ministry of Public Works were responsible for supervision.

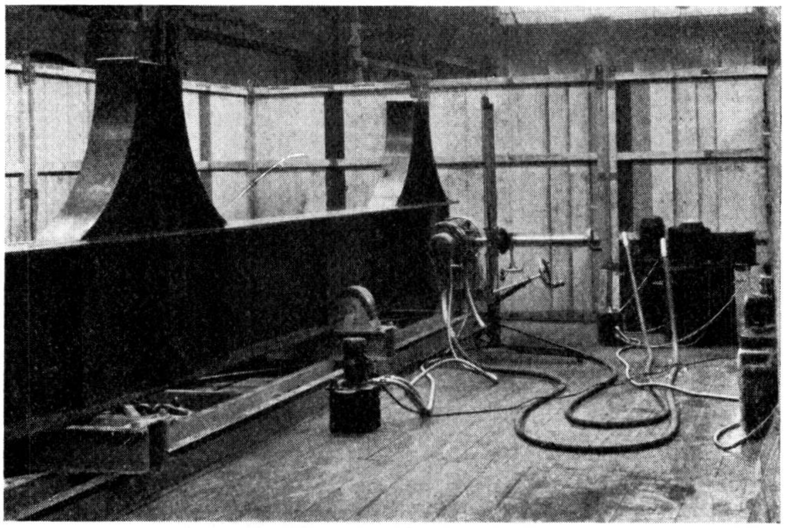

Fig. 13.

Finally Fig. 14 and Table VII give particulars of the principal welded road bridges in Czecho-Slovakia completed up to the present time.

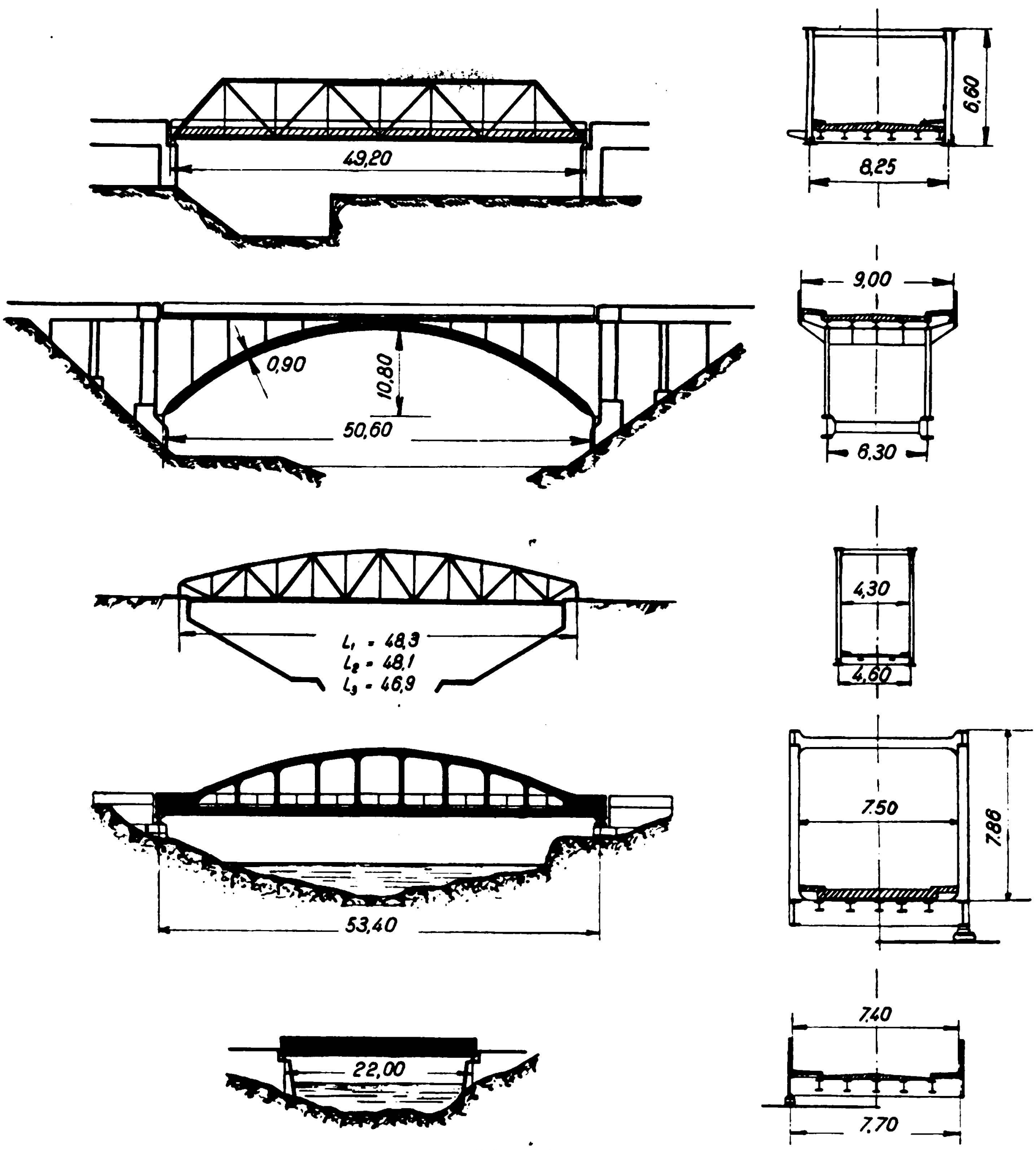

Fig. 14.

Table VII.

Bridge No.	Year of Construction	Span in m	Loading	Weight in tonnes	Construction
1.	1931	49.20	Class I	145.0	Škoda Works, Pilsen
2.	1933	50.60	„ I	111.0	„ „ „
3.	1933	22.00	„ I	37.6	„ „ „
4.	1934	48.30	„ III	52.0	Českomaravska-Kolben – Daněk
		48.10	„ III	52.0	Brno-Kralovopolská
		46.90	„ III	49.1	Škoda Works, Pilsen
5.	1936	53.40	„ I	157.0	„ „ „

Note: Class I corresponds to a uniformly distributed live load of 500 kg/m^2 or to a road roller weighing 22 tonnes.

Class III corresponds to a uniformly distributed live load of 340 kg/m^2 or to a wagon weighing 4 tonnes.

IIIc 8

The Calculation of Welds.

Berechnung der Schweißnähte.

Le calcul des soudures.

Ir. N. C. Kist,
Professor an der Technischen Hochschule in Delft, Haag.

In a brief verbal summary of his paper in Group IIIc on "The Calculation of Welds Assuming Constant Energy of Change of Shape" the author emphasised that in any statically indeterminate connection the direction of the force transmitted by a weld should be determined by reference to the plasticity theory.