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# IIIa 1

## General Considerations on Welding.

## Allgemeine Betrachtungen über das Schweißen.

## Considérations générales sur le soudage.

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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 $\sigma_B = 58\ \text{kg/mm}^2$ and a yield point $\sigma_F = 44\ \text{kg/mm}^2$ 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 $\sigma = \frac{M_{max} - 0{,}3\,M_{min}}{W_n} \leqq \sigma_{zul} \leqq 14\ \text{kg/mm}^2$. 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 $\sigma_o$ line shown in Fig. 1, referred to lower stress value $\sigma_u$. The amplitude $\sigma_S = \sigma_O - \sigma_U$ is determined from the distance between $\sigma_O$ line from a line inclined at $45^0$. 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 $\sigma_S$ 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 $\varphi$. 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" $\alpha$ is introduced in addition to the vibration factor $\gamma$ appertaining to an unjointed member, in order to allow

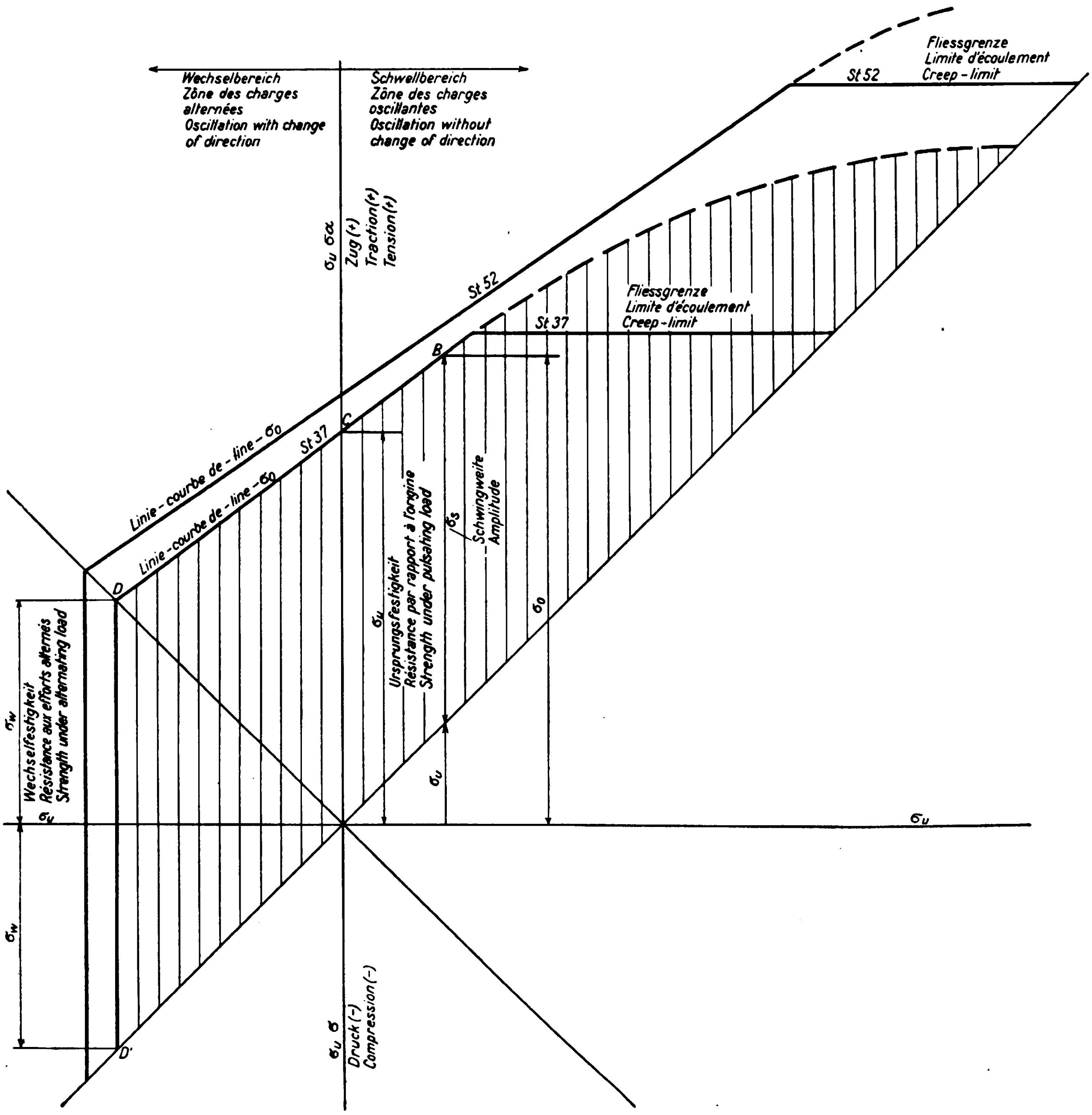


Fig. 1.

Curve of fatigue strengths $\sigma_o$ 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 $\sigma_1 = \frac{\gamma}{\alpha} \cdot \frac{\max M_I}{W_n} \leqq \sigma_{zul}$ 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 $\gamma$ and again multiplied by the coefficient $\alpha$ according to the nature of the construction and that of welding. Thus the calculation is performed on the basis of imaginary stresses $\sigma_I$, 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 $\gamma$ and $\alpha$ applicable to railway bridges are shown in Tables 1, 2 and 3 of the "Provisional Regulations for Welded Solid-Webbed Railway

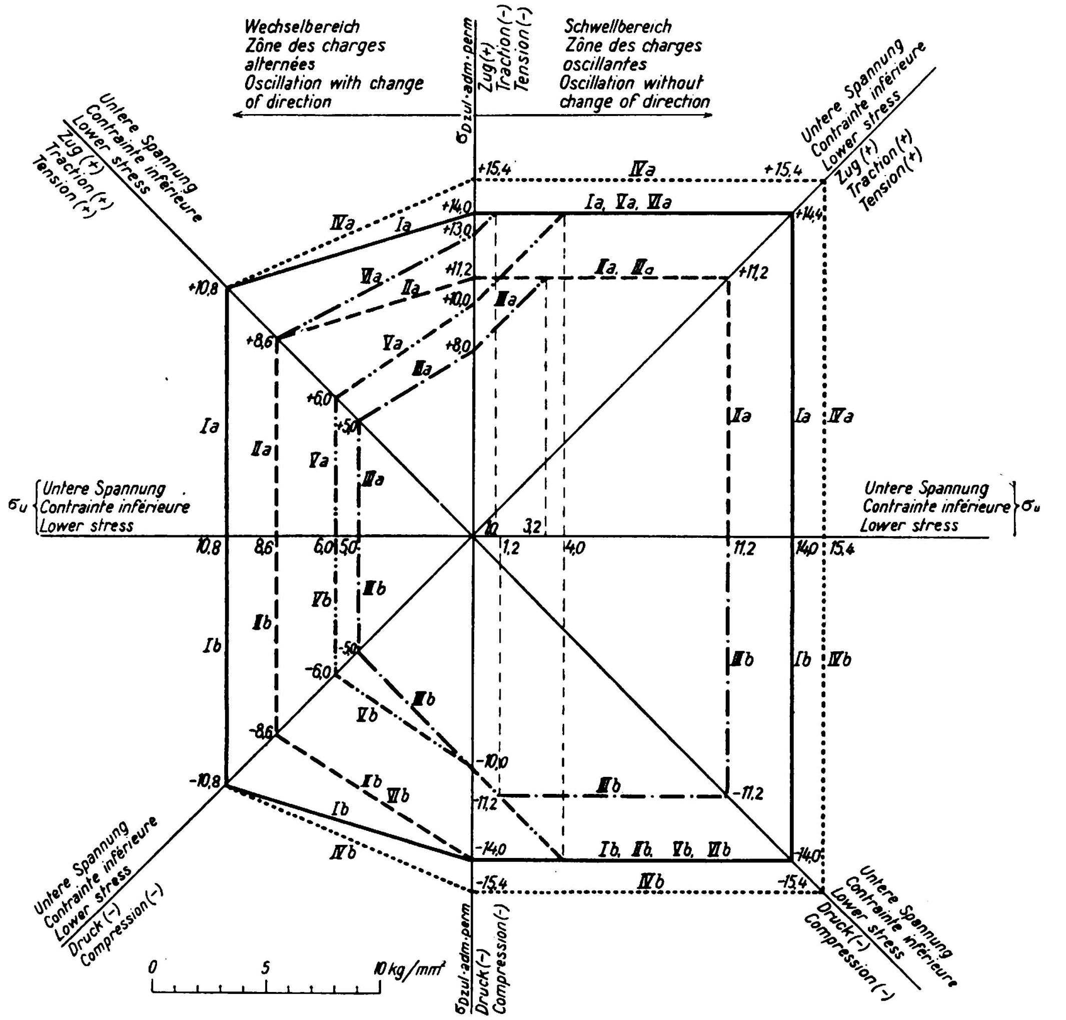


Fig. 2a.

Diagram of permissible stresses $\sigma_{D\,zul}$ 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 $\gamma$, which depend on the ratio $\frac{\min S_I}{\max S_I}$ or $\frac{\min M_I}{\max M_I}$ may be

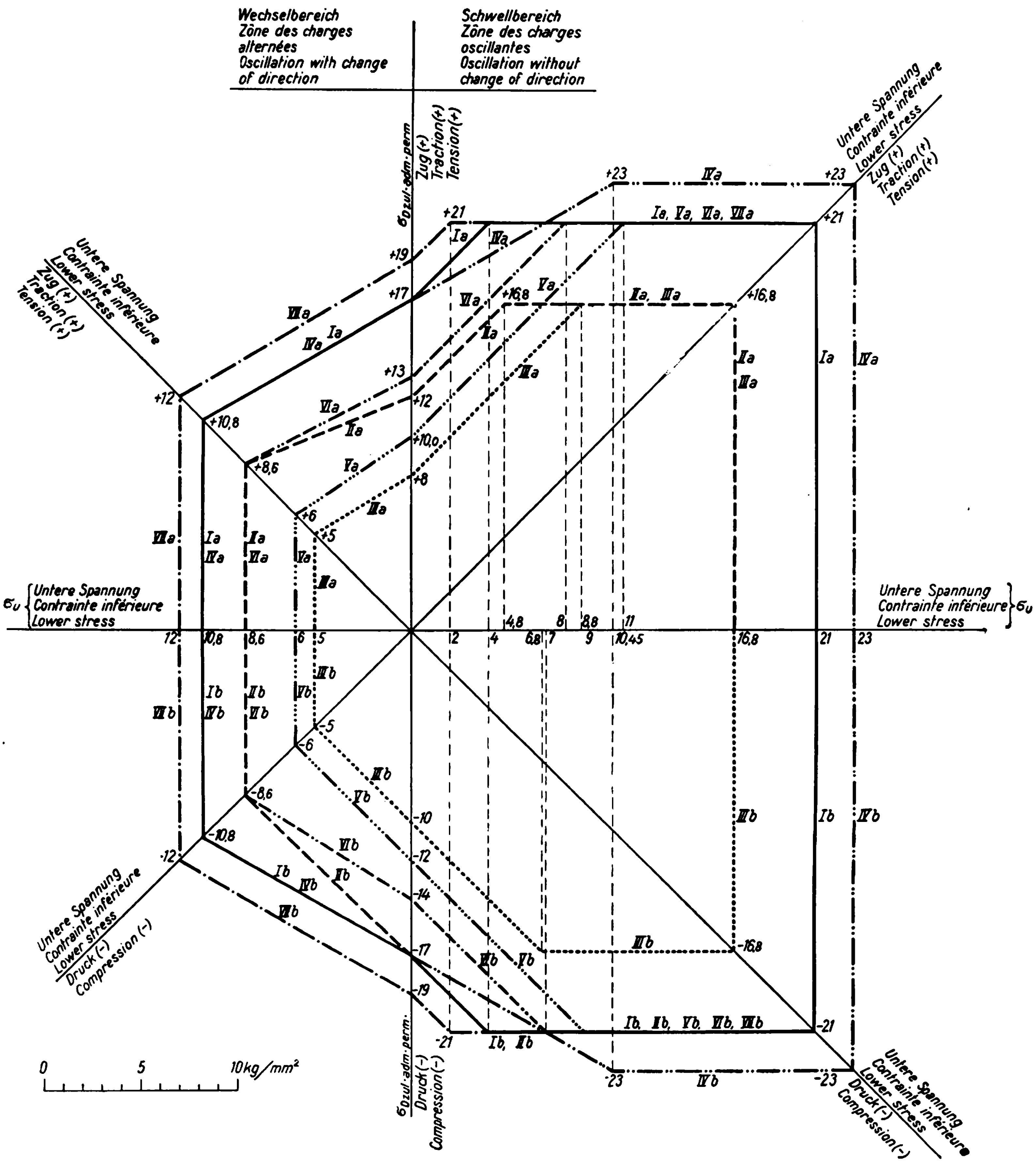


Fig. 2b.

Diagram of permissible stresses $\sigma_{D_{zul}}$ 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 $\alpha$ 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 $\sigma_{max}$ 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/mm² 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.)

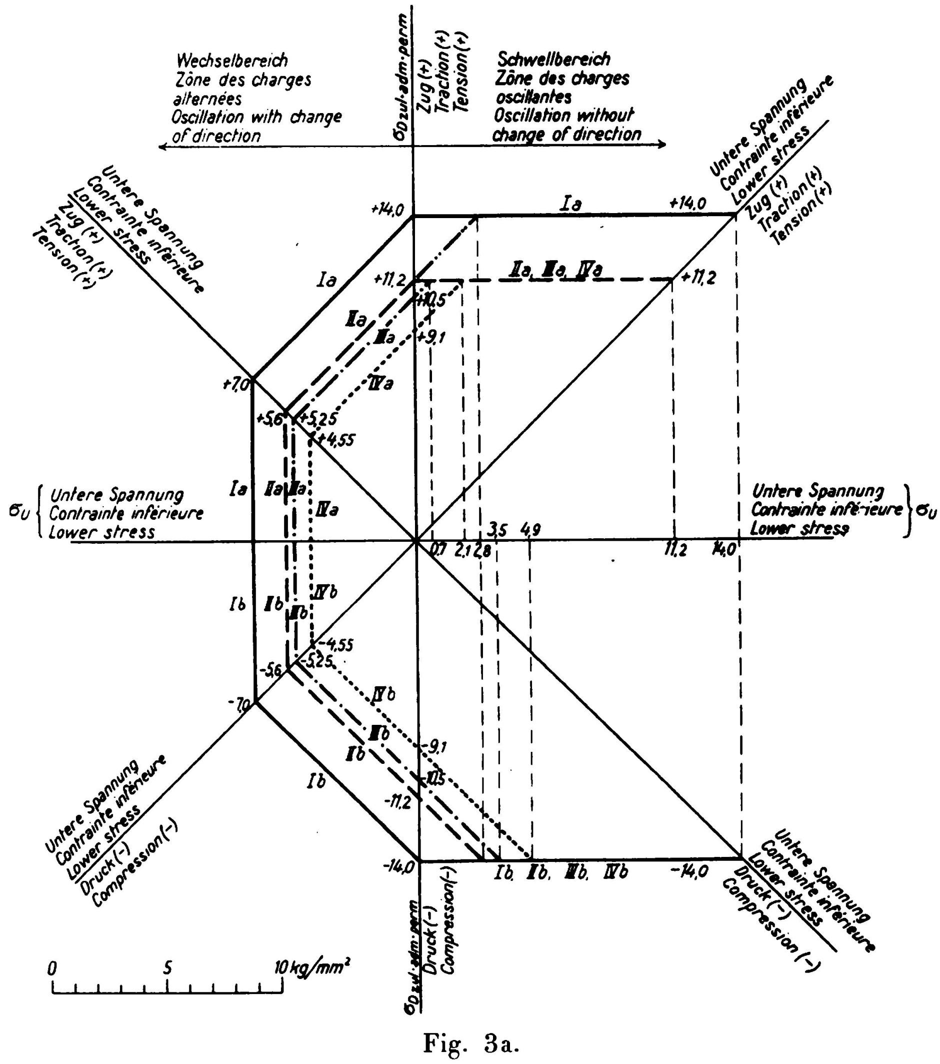


Fig. 3a.

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

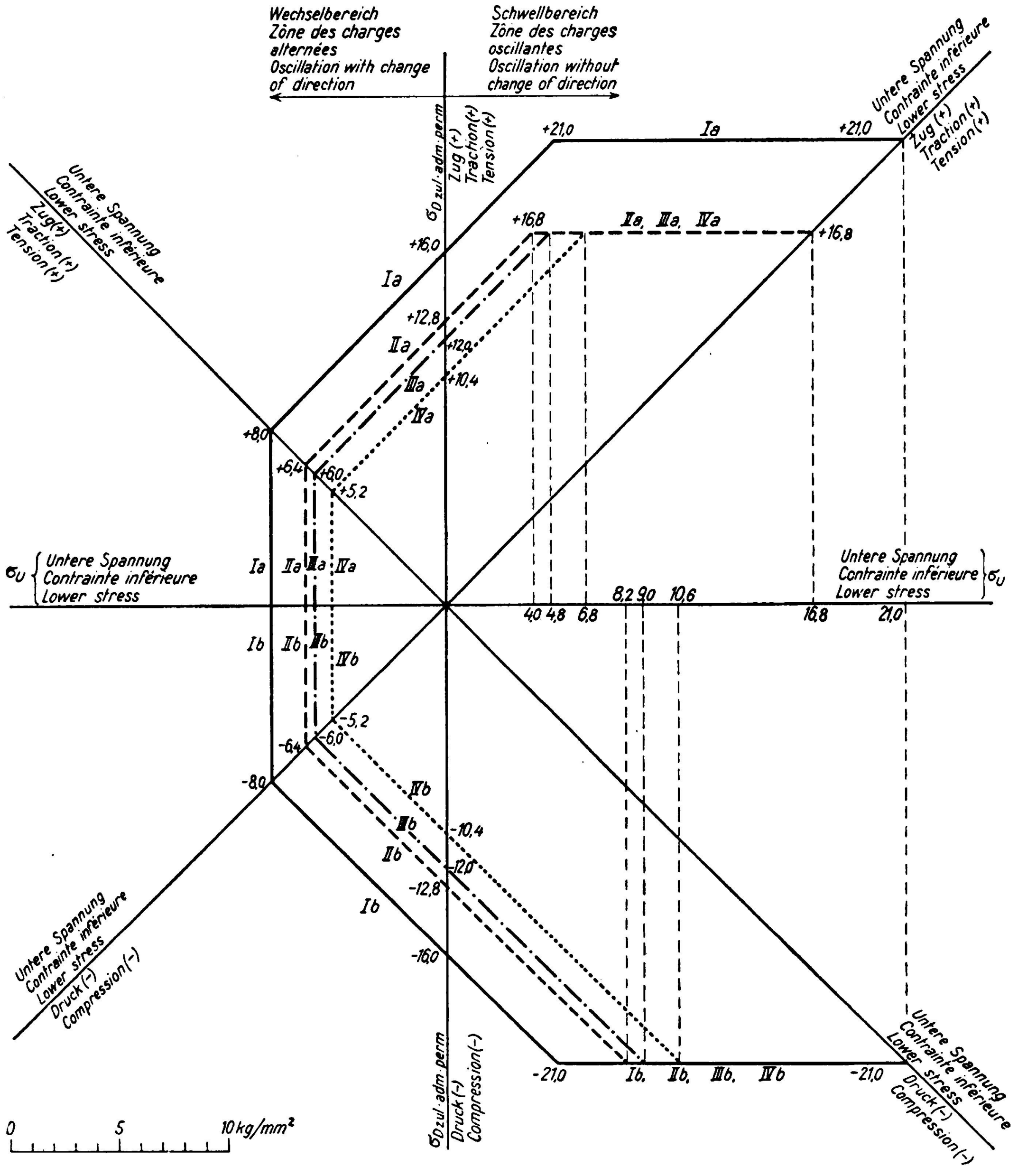


Fig. 3b.

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.

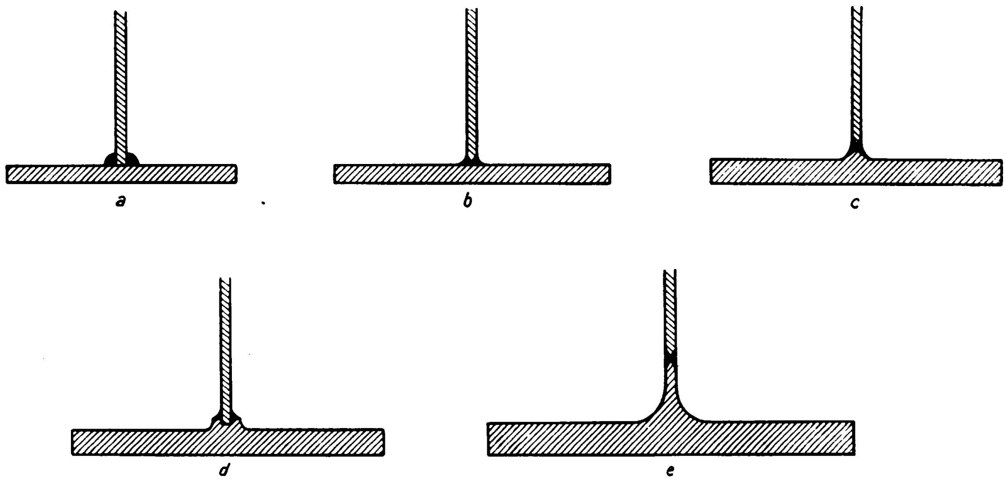


Fig. 4.

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.

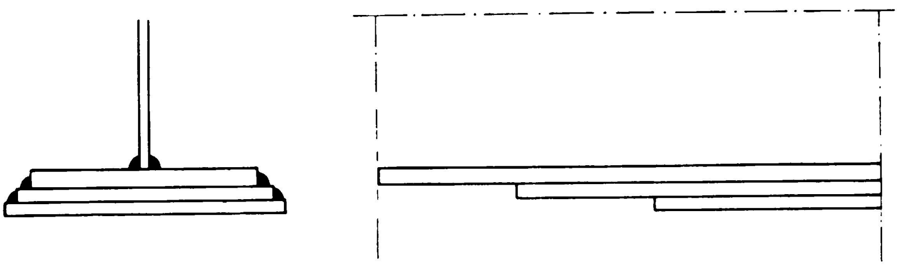

Fig. 5.

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

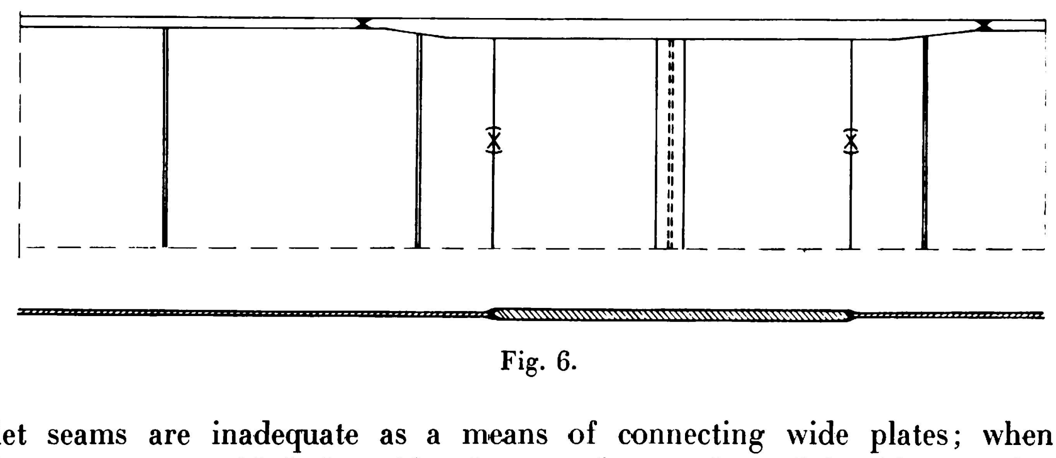

Fig. 6.

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

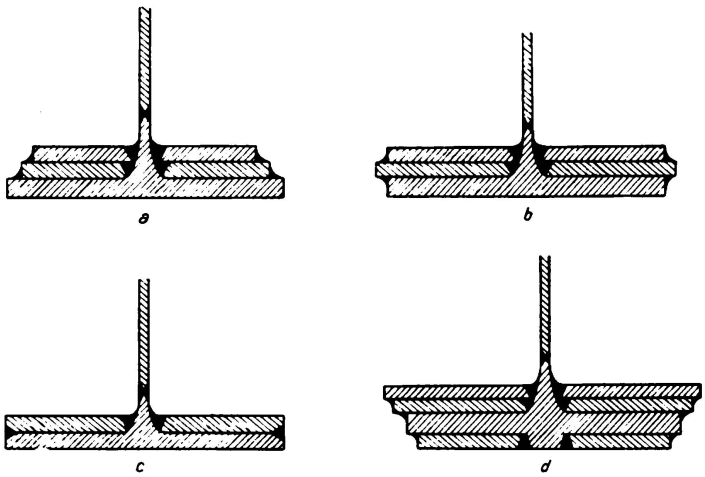


Fig. 7.

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

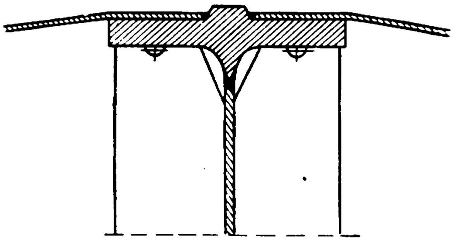

Fig. 8.

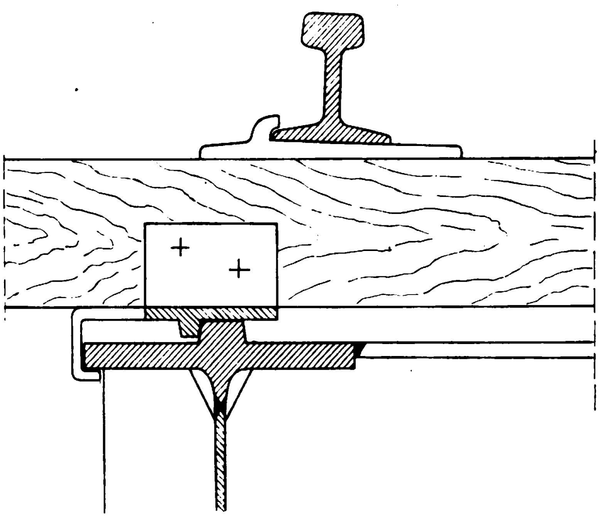

Fig. 9.

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).

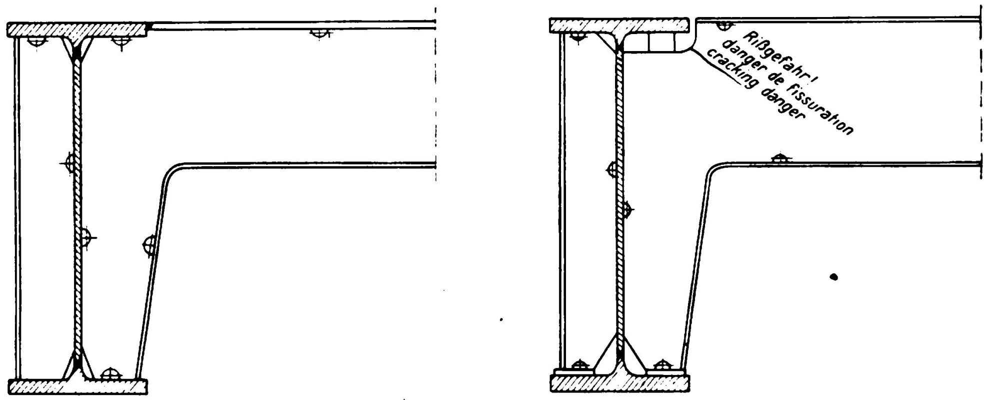


Fig. 10.

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.

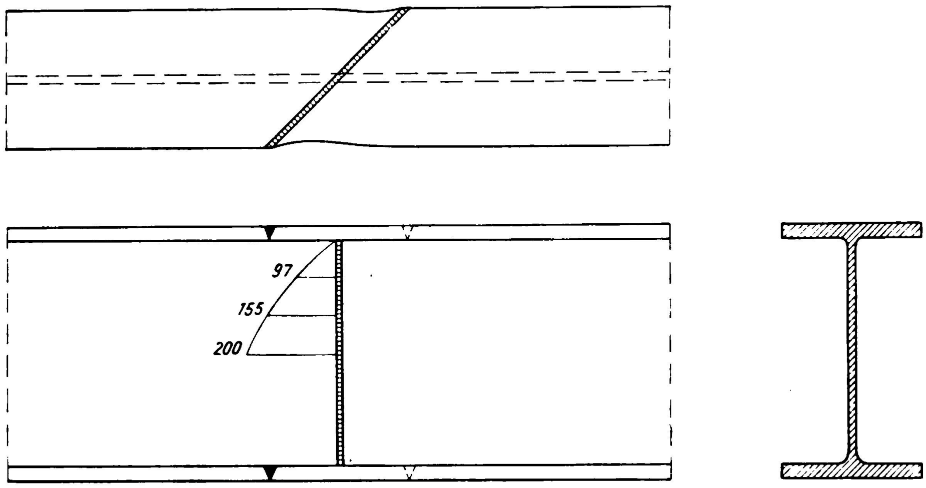


Fig. 11.

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.