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IIIa 1

The Influence of Frequently Alternating Loading on Welded Structures.

Einfluß häufig wechselnder Belastungen auf geschweißte Bauwerke.

Influence des variations de charge répétées sur les constructions soudées.

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A. Introduction.

At the 1st Congress in May 1932 in Paris I had the honour to read a paper concerning the calculation and design of welded structures. In that paper I treated the prevailing conditions and experiences made with welding up to that time in Germany. The calculation was simply based on mere static tests. In my summary[1] I stated that tests had shown that calculations carried out according to the German regulations supply results with sufficient safety in respect to static efforts only: Whether this safety suffices also for dynamic efforts, will be established by the results of tests which are still being carried out at present.

A clear opinion existed already at that time, that only extensive trials carried out with Pulsator-machines in connection with swing-bridges would throw light on the completely unsolved problems. Under the guidance of Dr. ing. h. c. *Schaper*. Director of the German State Railways, the Board of Administrators arranged for such dynamic fatigue tests, spending about 50.000 Mk. These tests were concluded in 1934.

Regarding these tests I have written at length in the 3rd vol. of Publications under the heading: Results of fatigue strength tests on welded connections. In my complementary remarks on page 263 of the 3rd vol. of Publications I pointed out that the permissible stresses for welded connections were increased by the members of the Committee in their final meeting which took place in August 1935 in Friedrichshafen. The formula for calculation also received a modification in the final regulations. As regards the valuation of test results it suffices to refer to the 3rd vol. of Publications. But I consider it a necessity

[1] See Preliminary publication: "Fatigue tests on welded connections", Berlin 1935 V. D. I. and *Kommerell:* "Explanations on the regulations relating to welded steel structures and their design". Part I "Structural Steel Engineering", Berlin 1934. Part II "Welded Plated Railway Bridges". Berlin 1935. W. Ernst and Sons, Editors.

once again to summarize the test-results and to explain the conclusions drawn from the final regulations.

The Illustrations and Tables marked V relate to the Regulations of the German Government for Welded Plated Railway Bridges.

B. Definitions.

The purpose of fatigue tests is to establish the value of the resistance which a test bar can stand under frequently repeated loadings.

σ_u indicates the lower stress values (pre-stressing)

σ_o indicates the upper stress values (stress limit after $n \times 10^6$ repetitions of loading)

(for tension (+), for compression (—).

If σ_u and σ_o possess the same sign, we speak of oscillation of loads without change of direction (surging loads), and if σ_u and σ_o have different signs it is for pulsations of loads with change of direction (alternating loads). If we wish to emphasize especially that a stress is only tensile (+) the indicator (z) is added and for compression (—) the indicator (d) e · g · as under

σ_{oz} upper stress, in tension

σ_{ud} lower stress, in compression.

To make these matters more comprehensive we employ the method introduced by *Weyrauch*[2] used ever since in Germany. On the axis of abscisses are marked the lower stresses σ_u. Through the origin O of the system two lines are drawn under 45^0. For any point A on these lines the ordinates are consequently equal to σ_u. The ordinates for tensile stresses (+) are marked above and for compressive stresses (—) below the horizontal axis. The ordinate for B indicates the upper stress σ_o for $n \times 10^6$ repetitions of loading (see fig. 1).

Not all test bars will fait at the same number of loading repetitions, therefore the so-called Woehler-line is used to derive the value of the upper stress which would exist if the test piece had stood 2×10^6 pulsations. The values of the test results given in the report of the Board of Administrators were recalculated with this figure of loading repetitions. This figure also forms the basis of all subsequent explanations. For 2×10^6 pulsations the upper stress limit $\sigma_o = \sigma_D =$ fatigue strength. The distance between the σ_o-line and the σ_u-line under 45^0 represents in a clear way a most important value which is called the amplitude σ_u of oscillation. If $\sigma_u =$ o (origin) hence $\sigma_o = \sigma_u$ — original surge load strength. Should the compressive stress σ_u be equal to the tensile stress σ_o, we speak of alternating strength σ_w and if the higher stresses σ_o are all tension then the vertically hatched area in Fig. 1 indicates the range of oscillation, the horizontally hatched area in fig. 2 indicates the range of oscillation in case the compressive stresses σ_o are higher in value.

To the right of the axis of ordinates we have only stresses of one and the same sense and direction. This is the range of surging forces or oscillation of forces without change of direction (Schwellender Bereich), but to the left of

[2] *Weyrauch:* Die Festigkeitseigenschaften und Methoden der Dimensionenberechnung von Eisen- und Stahlkonstruktionen, Tafel IV, Fig. 66, Leipzig 1889, Verlag Teubner.

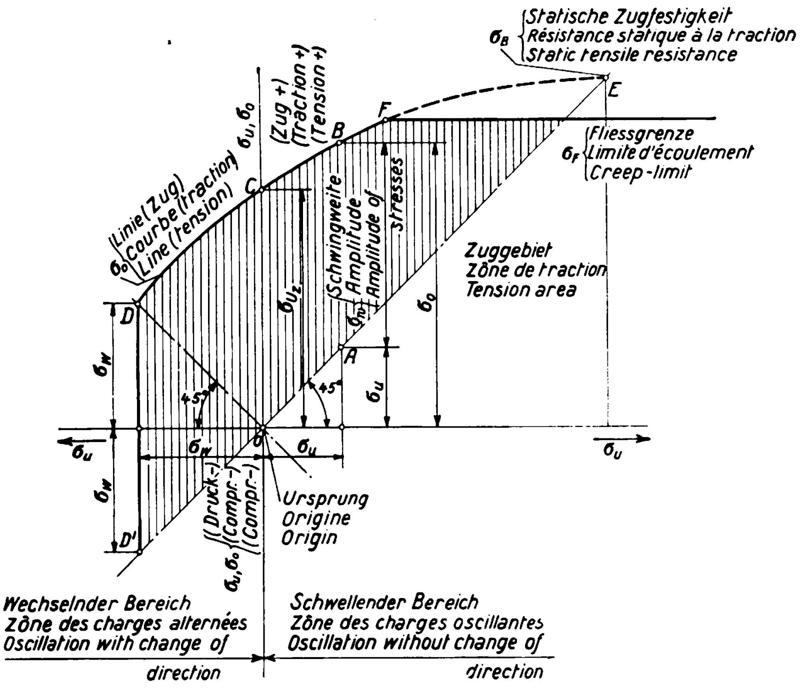


Fig. 1.

Curve of fatigue strength (σ_o-line) if σ_o represents tension.

the vertical axis lies the range of alternating forces or oscillation of forces with change of direction, where the stresses σ_o and σ_u have reverse signs.

The stresses σ_o for the tensile area in fig. 1 can only go up to the yield point

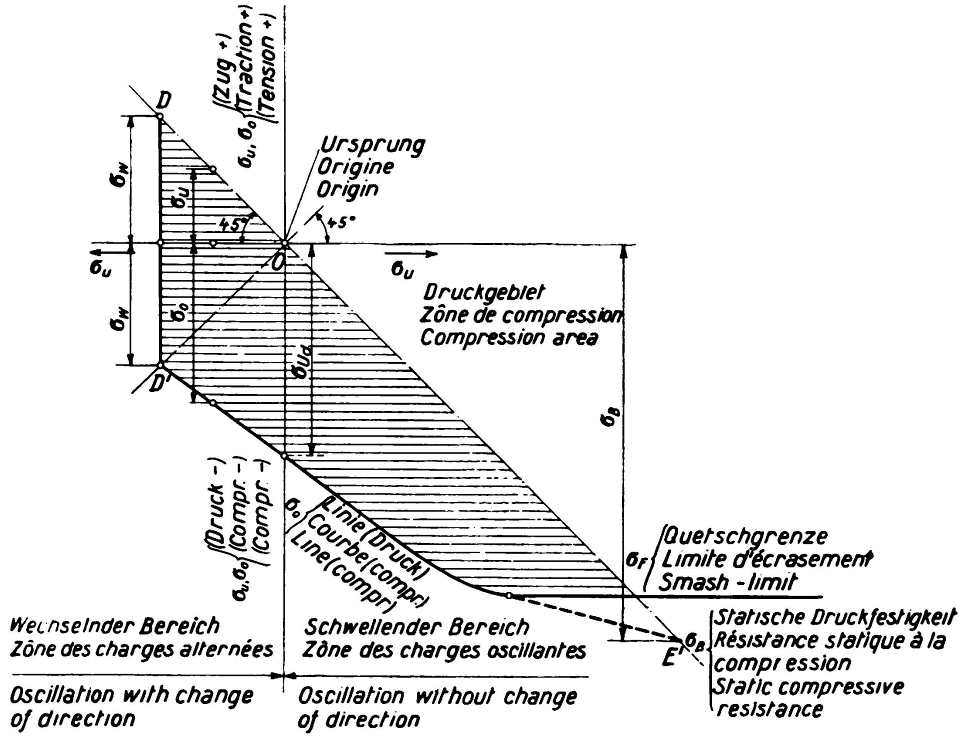


Fig. 2.

Curve of fatigue strength (σ_o-line) if σ_o represents compression.

σ_F and for the compressive area in fig 2 up to σ_{-F} respectively. (The absolute values of σ_F = yield point and σ_{-F} = crushing point fig. 2 can be assumed to be identical. Up to now only the range of (oscillation of forces without change of direction) surging loads in the tensile area has been properly explored. The study of the test results has shown with sufficient accuracy that for the area of tensile stresses the σ_o-line can be replaced by a straight line under an angle α. The angle α is in this case less than 45^0 and varies with the type of welding-seam (butt-weld, fillet-weld). From this it follows that for the tensile area (fig. 1) the amplitudes of oscillation become less the nearer we approach the yield limit. The few investigations (by *Graf*) into the range of alternating forces prove that we calculate rather unfavourably if we lengthen the straight-line diagram for σ_o down to the value σ_w for alternating strength. Accordingly the values σ_w, in the report of the Board of Administrators, have been defined with the angle α.

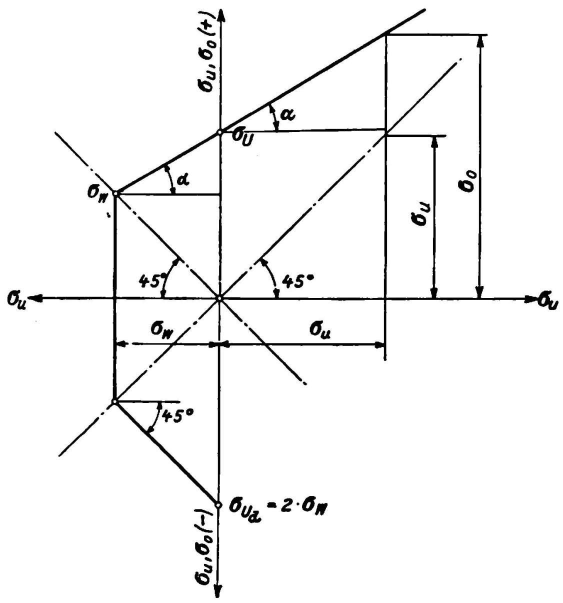


Fig. 3.

The angle α is expressed by the relation

$$\operatorname{tg} \alpha = \frac{\sigma_U - \sigma_W}{\sigma_W}$$

hence

$$\sigma_W = \frac{\sigma_U}{1 + \operatorname{tg} \alpha} \tag{1}$$

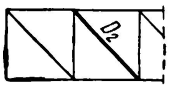
For the compressive zone we can assume with sufficient accuracy that the amplitude of oscillation σ_w has everywhere the same value $\sigma_w = 2\,\sigma_W$. This means that in this case the angle $\alpha = 45^0$. The surge load strength for the zone of compression has accordingly been laid down as $\sigma_{Ud} = \sigma_w = 2\,\sigma_w$. As mentioned previously the number of loading repetitions for endurance-strength tests is usually fixed to $N = 2 \times 10^6$. To obtain a clear conception as to the proper meaning of this figure we take the case of a single track railway bridge over which 25 trains pass daily. Naturally only the maximum stresses produced by the passing train in the various parts of the bridge are of interest in this case. Decisive, in general, is only one position of the train, the most unfavourable. In this particular case

$$\frac{2{,}000{,}000}{25 \times 365} = \sim 220 \text{ years}$$

are required to produce 2×10^6 changes of loading. As a rule and under normal conditions the train loads which pass over bridges are lighter than those loads on which the design was based. The life of such a bridge is of course shorter for other reasons (corrosion, considerable increase in rolling loads), but we are on the safe side in general if we are satisfied that the bridge stands 2×10^6 changes of loading of the most unfavourable kind.

The question had also arisen, previous to the publication of the new (BE) "Basis of calculation for steel railway bridges, 1934" if it would not be advisable to increase the permissible stresses for double track railway bridges in comparison to single track railway bridges. It has been found that the standard loadings prescribed for designing are under actual traffic conditions only very rarely realized for double track railway bridges. This would cause the fatigue strength for a smaller number of changes in loading in relation to the Woehler line to be higher. The following table gives for five alternative cases of loading for a railway bridge the difference $\Delta\sigma$ of actual stresses to those calculated.

Table 1.

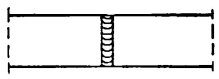
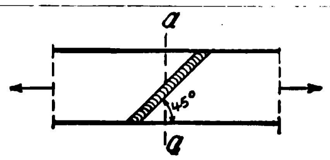
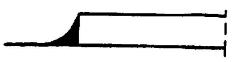
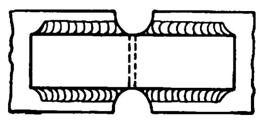
1	2	3	4	5	6	7
Span l m		member max stress in double track Railway Bridges in st. 37 I one track fully loaded, 2nd track not-loaded kg/cm²	II one track fully loaded, the other track loaded with 3,6 t/m kg/cm²	III one track fully loaded, the other track loaded with 8,0 t/m kg/cm²	IV one track fully loaded, the other track loaded with 2 Locos, type N kg/cm²	V Both tracks loaded accord to regulation and design kg/cm²
70	Lower cord	1080 $\Delta\sigma = 335$	1185 $\Delta\sigma = 215$	1310 $\Delta\sigma = 90$	1315 $\Delta\sigma = 85$	1400
	Diagonal D_2 in tension	1065 $\Delta\sigma = 335$	1160 $\Delta\sigma = 240$	1290 $\Delta\sigma = 110$	1310 $\Delta\sigma = 90$	1400
100	Lower chord	1100 $\Delta\sigma = 300$	1200 $\Delta\sigma = 200$	1330 $\Delta\sigma = 70$	1270 $\Delta\sigma = 130$	1400
	Diagonal D_2 in tension	1085 $\Delta\sigma = 315$	1185 $\Delta\sigma = 215$	1300 $\Delta\sigma = 100$	1290 $\Delta\sigma = 110$	1400

A decision has been reached, not to treat double track railway bridges differently from single track bridges in cases of entirely new bridge construction. But this question may become important in the case of strengthening old bridges. Similar considerations and views may also apply for road bridges.

C. The most important results of the report of the Board of Administrators[3] (Endurance-strength tests).

1) The values of endurance-strength (fatigue strength). The endurance-strength (fatigue strength) values σ_D as derived from test results are shown in the following table 2.

Table 2 (for steel 37).

1	2	3	4	5	6
No.	Type and nature of weld	Alternating strength σ_W (derived) for $2 \cdot 10^6$ loading repetitions kg/mm²	Surge-load strength Tension σ_{U3} for $2 \cdot 10^6$ loading repetitions kg/mm²	Compression σ_{Ud} (derived)	Reference to tables, report of Board of administrators
1	 Butt weld, root rewelded	11	18	— 22	table 5** of figures, line 2
2	Same as for 1 root not rewelded	8	13	— 16	table 5** of figures, line 1
3	 Butt weld as for 1 but under 45°	13	22	— 26	table 5** of figures, line 3
4	 light end-fillets with gradual transition from weld to plate	5,4	10,3	— 10,8	table 13** of figures, line 2
5	Full End-fillet welds, no tooling	3,4	6,5	— 6,8	table 13** of figures, line 3
6	 light sight fillet welds machining of terminates of fillets	6,3	12,0	— 12,6	table 13** of figures, line 6
7	Full side fillet welds, without machining of terminates of welds	4,2	8	— 8,4	table 13** of figures, line 5

[3] See *Kommerell:* Erläuterungen zu den Vorschriften für geschweißte Stahlbauten, 4. Auflage, Teil I Hochbauten, Teil II Vollwandige Eisenbahnbrücken. (Explanations to the regulations relating to welded steel structures. 4th Edition Part I Structural Engineering, Part II Plated Railway bridges.)

2) *General remarks on the figures of table 2, and flow of forces.*

Even the first trials with pulsator machines gave compared with the static strength (rupture strength $\sigma_B = 40$ kg/mm^2) remarkably low endurance-strength (fatigue strength) values (surge load strength $\sigma_U = 8$ kg/mm^2), this chiefly in the case of side-fillet welds. The reason for this lies in the flow and transmission of forces as will be explained subsequently.

For dynamically stressed welding connections it is of great importance to study the flow of forces very carefully. Peak stresses produced by sudden changes in cross sections, or by sharp corners of notchings, or by the unsuitable positions of joints for which the welding proves difficult, should on account of the "notching action" be fully avoided or at least their effects be reduced by skilled structural arrangements. All forces should be transmitted from one part to another in the shortest way and in a natural manner, avoiding distinct changes of direction.

In structural framework such peak stresses are not of the same importance as in structures dynamically stressed. It is for this reason that steel, on account of its plasticity after certain parts have been statically overstressed, and after exceeding the yield limit, is capable of reducing the peak stresses in such a way that previously less stressed areas of cross sections take over more load. The difference between statically and dynamically stressed parts will be obvious from the following test[4] (fig. 4). On the left in fig. 4 we see the result of a statically stressed flat steel piece.

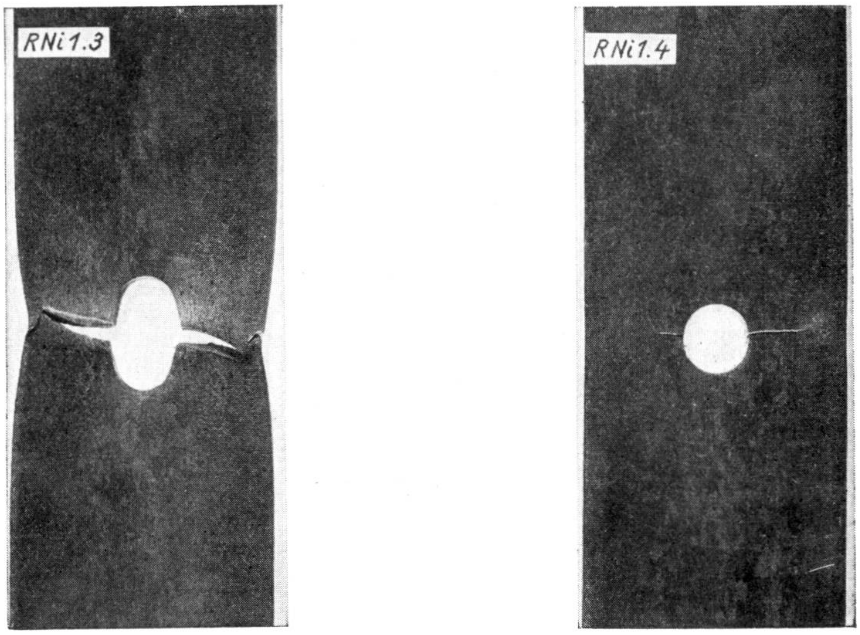


Fig. 4.

Static tensile test
tensile strength
$\sigma_B = 54.6$ kg/mm^2

Fatigue tensile test
surge load strength
$\sigma_U = 24.0$ kg/mm^2

Trials with perforated test bars.

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[4] See *Graf:* Über die Festigkeiten der Schweißverbindungen . . . Autogene Metallbearbeitung 1934, page 1 etc.

The drilled hole causes in a section through the centre an uneven distribution of stresses, forming peak stresses at the edge of the hole.

This test gave a tensile strength of $\sigma_B = 54{,}6$ kg/mm^2 whilst the dynamically stressed piece had a surge-load strength of $\sigma_B = 24$ kg/mm^2 only.

3) *Butt-Welds.*

a) At the outset of welding, particulary butt-joints were treated with caution but since a large number of fatigue tests in this connection have been made, the scepticism about this type of joint disappeared. Butt-welds, on account of their easy and natural flow of forces, proved superior to fillet-welds. To-day, butt-welds carried out with some care, give a surge-load strength of $\sigma_U = 18$ kg/mm^2 as high as for a pierced-flat.

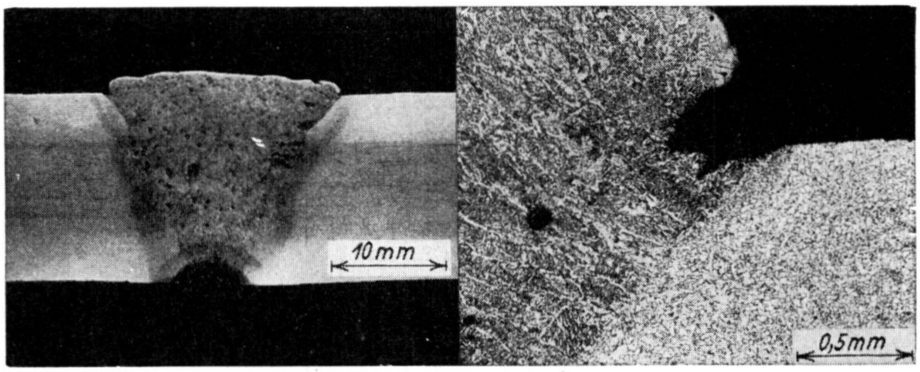


Fig. 5a.

Surge load strength $\rho_{Uz} = 10$ kg/mm^2
Static tensile strength $\sigma_B = 34$ kg/mm^2.

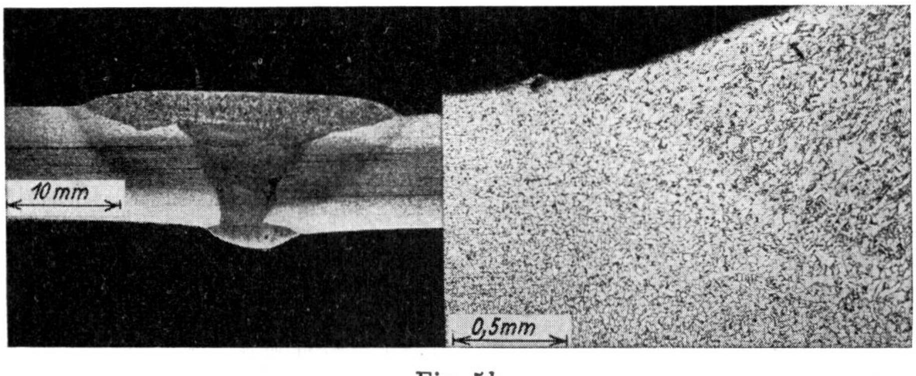


Fig. 5b.

Surge load strength $\rho_{Uz} = 18$ kg/mm^2.
Static tensile strength $\sigma_B = 37.5$ kg/mm^2.

Fatigue tension test with butt welds with and without pores and pitting at the transition zone.

The endurance (fatigue) strength is strongly dependent on the workmanship of the weld. In Fig. 5a a case is shown where the connection between weld-metal and parent-metal is very badly executed and the weld-metal itself is full of pores and blow-holes[5]. This specimen had a surge-load strength for the weld-metal

[5] See *Graf:* Über die Festigkeiten der Schweißverbindungen . . . Autogene Metallbearbeitung 1934, p. 4 and 5.

of $\rho_U = 10$ kg/mm^2 only (notching action), whilst the specimen shown in fig. 5b having no such drawbacks produced a surge-load strength of $\rho_U = 18$ kg/mm^2. In both cases the static tensile strength had values not much different from each other, but the higher value was also here for the weld of better workmanship.

b) The welding seams should only slowly increase in thickness and bulge only little over the surface of the parent metal. It is wrong to believe that thicker welds increase the endurance- (fatigue) strength, in fact the contrary is true.

c) It is important, too, that the root of the welded joint after being freed from slag, be carefully re-welded. The V-joint in fig. 6a (gas-fusion-welding) gave only a surge-load strength of $\rho_U = 12$ kg/mm^2 but the joint shown in fig. 6b with re-welded root attained a surge-load strength of 18 kg/mm^2. In both cases the static tensile strength

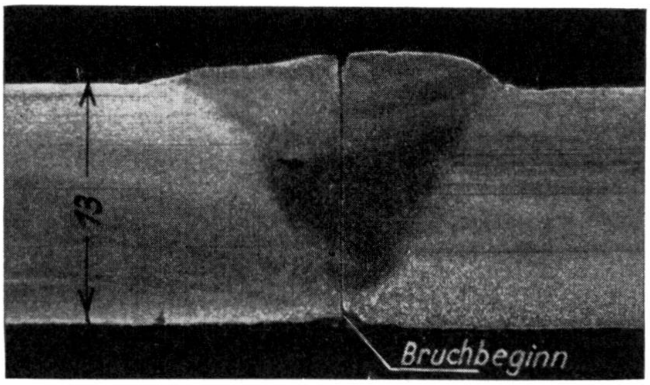


Fig. 6a.

Surge load strength $\rho_{U_z} = 12$ kg/mm^2.
Static tensile strength $\sigma_B = 38$ kg/mm^2.

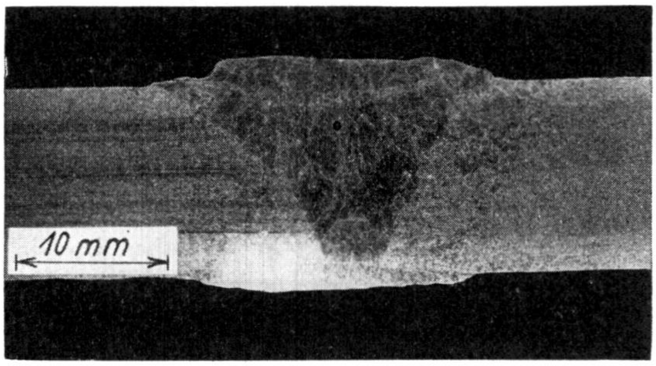


Fig. 6b.

Surge load strength $\rho_{U_z} = 18$ kg/mm^2.
Static tensile strength $\sigma_B = 38$ kg/mm^2.
Fatigue test with welds with and without rewelded roots.

was $\sigma_B = 38$ kg/mm^2. Many fatigue tests with X-shaped butt-joints proved the importance of removing all slag from the central-root (chipping, or cleaning with emery-wheel) and to re-weld it with a fine welding-wire (electrode), see fig. 7 showing a specimen with a surge-load strength of only $\rho_U = 10$ kg/mm^2. Joints in V-form proved in general superior to X-shaped joints (flange plates).

The V-joints of flange plates of bridge girders cannot always be re-welded, in which case it is definitely recommendable to pre-weld the bottom of the V with a thin wire (electrode), and to keep the plates as wide apart as possible to ensure good bondage also at the bottom of the joint. The joint in such a case should be situated at a place of small stresses.

d) The investigations of Prof. *Graf* have shown that the surge-load strength of butt-welds can be raised up to $\sigma_U = 24\ kg/mm^2$ if only the welds are planed and smoothened on both sides in the direction of the tensile forces. This fact indicates the importance of having a smooth and even surface free from notches if permanent strength is wanted. The surge-load strength can also be improved by careful grinding of the welds if care is taken to establish a gradual transition and the disappearance of unevenness.

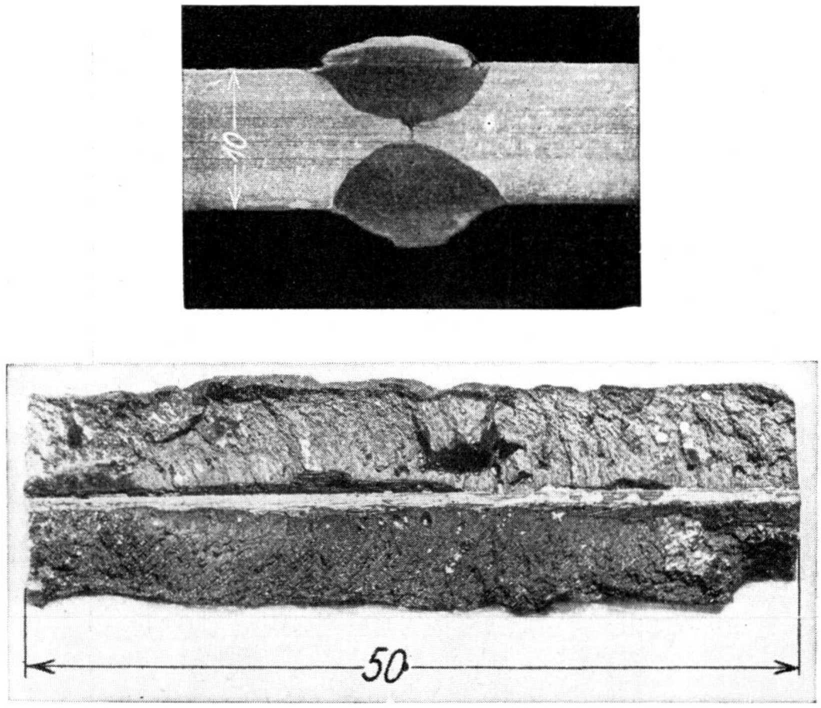


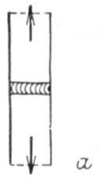
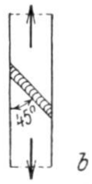
Fig. 7.

Deficient arc-welding of St. 37 X-shaped weld.

e) With the intention of increasing the fatigue strength of butt-welds it has been proposed also to use welding wires of superior strength (For jointing steel 37). Tests however have proved that it is useless to give the weld metal higher strength than the parent metal as the strength of the weld metal would never be fully used up and in most cases rupture does not occur in the weld metal but at the notchings between weld metal and parent metal.

f) An interesting test[6] has been carried out by Prof. *Graf* with a V-joint under 45⁰. From the following table below it will be seen that the surge-load strength of the V-joint can be improved.

[6] See: Autogene Metallbearbeitung 1934, p. 5.

		
Root of weld not rewelded	$\rho_U = 12$ kg/mm²	$\rho_U = 17$ kg/mm²
Root of weld rewelded	$\rho_U = 18$ kg/mm²	$\rho_U = 22$ kg/mm²

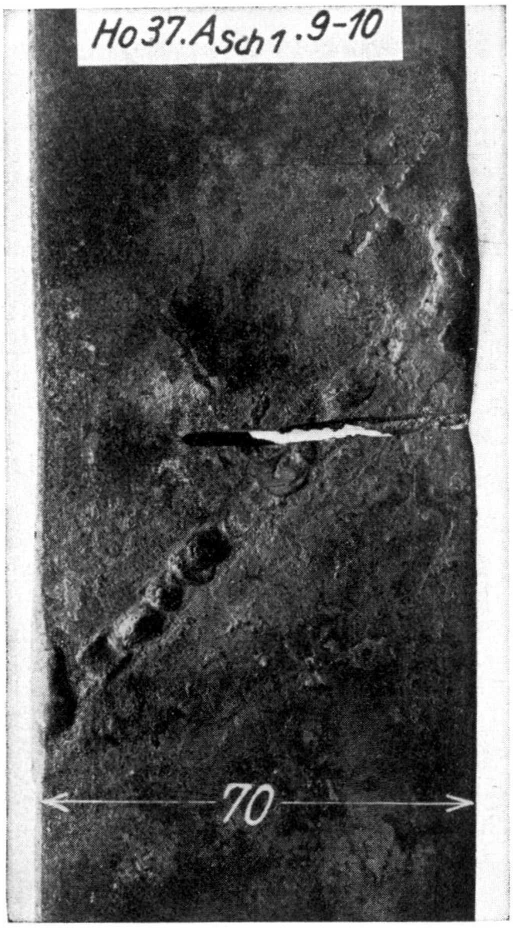


Fig. 8.

(root not rewelded.)

Fig. 9.

(root rewelded.)

Gas-fusion welding (oblique V-shaped weld) of St. 37 with frequently repeated tensile loading.

In both cases fractures started at places with fine notches. The section right angled to the direction of the acting force (case b) consists for the major part of parent metal in which case fine notches in the weld metal have a considerably smaller influence than in the case marked a. The fatigue strength for an oblique joint can still be increased by planing and smoothening the surface of the welded joint.

g) For the purpose of comparison Prof. *Graf* gives the following values of surge-load strength of unwelded flat irons

a) for steel 37 with rolling skin but without hole

$$\sigma_U = 25 \text{ to } 31 \text{ kg/mm}^2$$

b) for steel 37 with rolling skin and hole

$$\sigma_U = 16 \text{ to } 21 \text{ kg/mm}^2$$

For riveted joints the fatigue strength decreases in value on account of the bearing strength of rivet holes. This, particularly in a case where a coat of paint lies between the steel pieces. The test house of Dahlem has found for such cases that the surge-load strength of about $\sigma_U = 15$ kg/mm^2. For butt-welds with rewelded roots and dense, compact weld metal, however, the following surge-load strengths were obtained:

$\rho_U = 18$ kg/mm^2 for joint right angled to the direction of forces and

$\rho_U = 22$ km/mm^2 joint under 45^0 to the direction of forces.

It is noteworthy that with carefully welded butt-joints the same upper values of surge-load strength are reached as for flat steel pieces with rolling skin and hole, but up to now even machined butt-welded joints have not proved as strong as flats without hole. Tests carried out in Stuttgart indicate that the surge-load strength of butt-welded joints has a bigger variation between the lower and upper stress values compared with flat irons with and without holes; for ordinary untreated butt-welds the following surge-load strengths were found:

gas-fusion weldings for plates 10 to 26 mm thick of steel 37

$$\rho_{Uz} = 12 \text{ to } 18 \text{ kg/mm}^2 \text{ (5 specimens)}$$

electric-arc weldings for plates 10 to 16 mm thick of steel 37

$$\rho_{Uz} = 9 \text{ to } 18 \text{ kg/mm}^2 \text{ (12 specimens).}$$

The surge-load compressive strength for butt-welds can be assumed to be the same as the stresses at the yield point for the parent metal, hence $\rho_{Ud} = 24$ kg/mm^2.

4) a) *Side fillet welds.*

It has been found that the flow of forces for side fillet welds is less favourable compared with butt-welds. The lines of equal stresses (fig. 11) are crowded together on entering the joint plates and deviate from the main direction into the side fillets. When entering the connection plate a change in the direction of forces and at the same time lateral bending of the members so connected can occur. A strong decrease of the fatigue strength was found in cross-welds of test pieces worked out of the full, in addition to this we have a strong notching action at the transition of the weld metal to the parent metal. All these influences together increase the difference between static strength and fatigue strength for side fillet welds compared with butt welds. The static strength of side fillet welds of average workmanship is about the same as for statically stressed butt-welds, but the fatigue strength is considerably less. The test piece shown right in fig. 11[7] gave a surge-load strength of

$$\rho_U = 9—0.5 = 8{,}5 \text{ kg/mm}^2 \text{ compared}$$

with a tensile strength of $\sigma_B = 41{,}2$ kg/mm^2

[7] See *Graf:* Über die Dauerfestigkeit von Schweißverbindungen. Stahlbau 1933. p. 84 und 85.

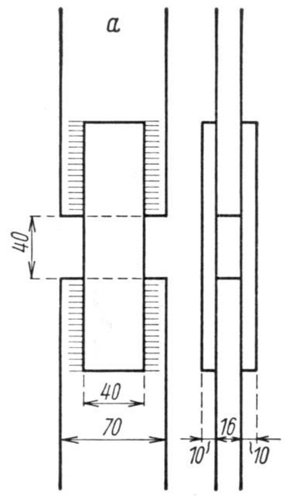


Fig. 10.

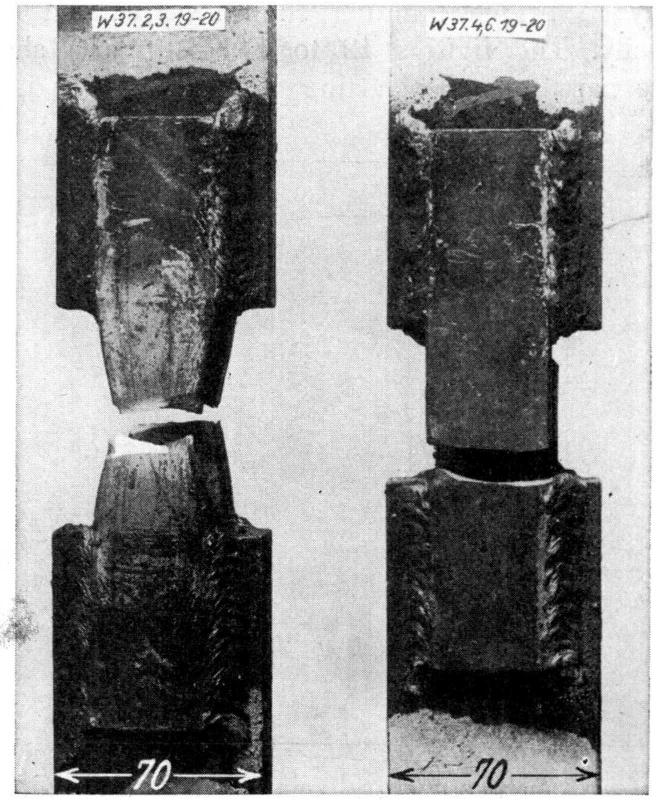


Fig. 11.

Tensile tests for side fillet weld connections acc. to Fig. 17. Arc-welding, St. 37, left after static tensile test, right after fatigue test.

for the statically stressed specimen shown left in fig. 11. The irregular transition between parent metal and weld (notching action) proves a great disadvantage (fig. 12). In structural framework side fillet welds can be employed with safety but not so for dynamically stressed connections where the permissible

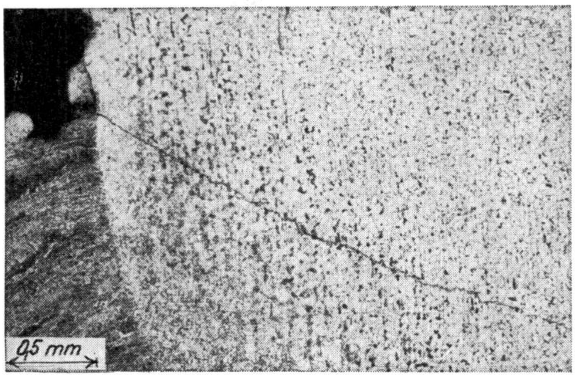


Fig. 12.

stresses have to be reduced for welds and members. The intensity of stresses in the ends of side fillet welds is very high, forming a wave of stresses, they are sometimes reduced by local plasticity of the material. Therefore all fillet welds chiefly in bridge building should be carried out with weld metal of high plasticity. The figures 12 to 14 (Stuttgart) show clearly that rupture starts in the parent metal (starting at the transition of parent metal to weld metal).

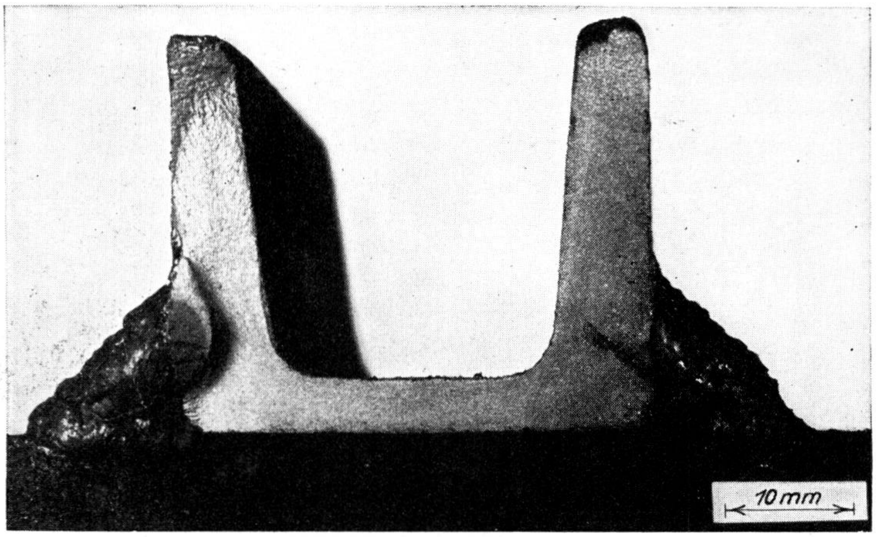


Fig. 13.

Therefore such structural elements having side fillet welds should not be stressed higher than compatible with the safety of the joint.

b) The deep inroad of the weld metal into the parent metal causing impor-

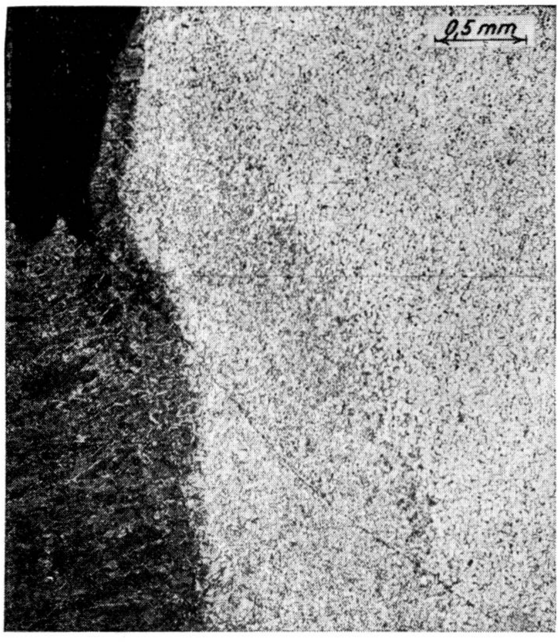


Fig. 14.

tant changes in the texture can produce strong notching actions, not only locally but over the whole zone of contact. For dynamically stressed side fillet welds a deep inroad of the weld metal into the parent metal should be avoided. It is further necessary that the welder keeps to the prescribed measurements when doing side fillet welds in bridge building or structural framework.

c) It is important that the welding of the fillet root be carefully executed.

d) The same remarks concerning the strength of weld metal mentioned for butt-welds under 3e apply for side fillet welds.

e) Tensile fatigue tests for side fillet welds have proved that the parent metal and not the weld metal fractures (in the contact area) if the following ratio

$$\frac{\text{Stress in weld metal}}{\text{Stress in parent metal}} = \frac{\rho}{\sigma} = 0.5 \text{ applies,}$$

but if the ratio $\frac{\rho}{\sigma} = 1$ or more, then fracture occurs in the weld metal. The surge-load strength ρ_U of fillet welds also increased with the length of the fillet. For a ratio $\frac{\rho}{\sigma} < 0{,}5$ the surge-load strength ρ_U increases but little. The ratio $\rho = 0.5$ indicates that the cross section of the weld is double the sectional area of the welded member. Beyond this ratio, the strength is ruled by the sectional area of the member and no longer by the sectional area of the weld. Therefore it is the member that fractures in the fatigue test and not the weld metal. Similar tests carried out in Dahlem and Dresden with swing bridges for side fillet welds have proved that for a ratio $\frac{F}{F_{Schw}} = 0{,}40$ to $0{,}83$ the members or the fish plates fracture at the ends of the welds and not the weld itself as

$$\frac{\rho}{\sigma} = \frac{\frac{S}{F_{Schw}}}{\frac{S}{F}} = \frac{F}{F_{Schw}},$$ [8] hence the ratios $\frac{F}{F_{Schw}}$

have the same meaning as the values $\frac{\rho}{\sigma}$.

Fractures at the end of the side fillet welds were the rule, even if [-sections were used instead of flat irons (Fig. 12, 13, 14 Stuttgart).

f) The wave of stresses at the ends of side fillet welds increase with the width B of the fish plate. In consequence ρ_U will be less if the width of the fish plate increases, but its thickness remains constant

Width of fish plate B	25	40	70 mm
ρ_U	10	9	7 kg/mm²

(see Stuttgart tests)[9].

Out of this the intention may arise to provide for dynamically stressed members a number of small fish plates, instead of one single but wide fish plate only. Still more recommendable, however, would be to choose fish plates of sufficient thickness.

g) To establish a more favourable flow of the forces, the front ends of the fish plates, in the above mentioned connections ($B = 70$ mm, $\rho_U = 7$ kg/mm²)

[8] and [9]. See: Fatigue strength tests with welded connection, Berlin 1935 V. D. I. Dauerfestigkeitsversuche mit Schweißverbindungen, Berlin 1935, V. D. I.-Verlag.

were slotted. This arrangement did not give a considerable increase in surge load strength compared with unslotted fish plates. The surge-load strength for slotted fish plates was $\rho_U = 8$ kg/mm^2.

h) For fish plate connections the side fillet welds should never be carried across the joint. For dynamically stressed fish plate connections it is also of great disadvantage if the opposite ends of the welds are too close together. Such an arrangement would decrease the fatigue strength of the fish plates on account of stress waves being produced in the sides of the fish plates above the gap of the joint. The following results of tests carried out in Dahlem and Dresden will illustrate these conditions[9].

Test Serial No.	Distance of weld ends over the gap of the joint	ρ_u kg/mm^2	ρ_o kg/mm^2	Number of loading repetitions endured 10^6
VI (St. 37) VIa (S II)	5	i. M. 8	16	0,30 to 0,51
VI E (St. 37) VIa E (S II)	50	i. M. 8	17	1,06 to 1,47
VI (St. 37) Specimen Da 4	5 but gradual transition due to machining	8	16	2,10

Test VI (St. 37 Da. 4) shows the favourable influence due to gradual transition between weld metal and parent metal.

A similar test was carried out by *Bierrett*[10] for which the fatigue strength was as follows

specimen, raw, not machined (fig. 15) $\rho_o = 8{,}5$ kg/mm^2 with 2 kg/mm^2 pre-stressing

Amplitude of stress $\rho_w = 6{,}5$ kg/mm^2

After machining weld ends (fig. 16) $\rho_o = 12{,}5$ kg/mm^2 with 2 kg/mm^2 pre-stressing

Amplitude of stress $\rho_w = 10{,}5$ kg/mm^2.

With a pre-stressing of 10 kg/mm^2 was found for untreated weld ends $\rho_o = 16$ kg/mm^2, hence an amplitude of oscillating stresses of $\rho_w = 6$ kg/mm^2. These two values are only suitable the comparison, since the fish plates were machine sheared only, which is not favourable for fatigue tests.

i) Arc-welded connections of steel 37 showed a considerable increase in surge-load strength from $\rho_U = 9$ kg/mm^2 to $\rho_U = 11$ kg/mm^2 if the gap of the joint was widened from 30 to 200 mm. The lines of equal stresses are straighter in this case, but the application of wide gaps between jointed pieces in structures will be very restricted.

[10] *G. Bierrett:* Die Schweißverbindung bei dynamischer Beanspruchung. Die Elektroschweißung, April 1933, Nr. 4.

k) Fatigue-strength tests with welded plate girders have shown that the fatigue strength of interrupted (not-through) fillet welds between flange and web is smaller than for through-fillet welds. The explanation for this is that a notching-action is introduced at the beginning and end of each welding strip due to the sudden change in cross-section. In bridge construction, particularly when sleepers are resting directly on welded plate-girders it is advisable to provide through-welds, and the same applies also for welded crane gantries. For structural building framework the conditions are quite different. Without hesitation interrupted fillet-welds between flange and web can be employed in structural framework, provided that no other reason (e · g · corrosion) demands through-welds.

5) *End-fillet welds.*

a) As regards the deep inroad of the weld metal into the parent metal the same applies as under 4b, and as regards careful welding of fillet roots 4c applies, and further the recommendations given under 3e concerning the quality of the weld metal apply also.

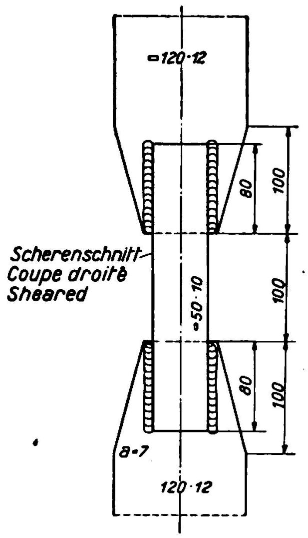


Fig. 15.

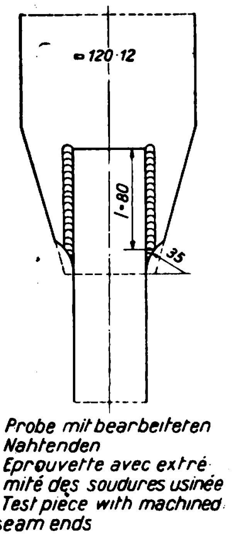


Fig. 16.

b) End fillet-welds as shown in fig. 17 and 18 start to fracture at r and subsequent fractures develop at s. The figures following are surge-load strengths attained in tests:

End-fillets { Gas-fusion welds fig. 17 and 18 $\rho_U = 11$ kg/mm²
Electric arc welds fig. 17 $\rho_U = 7$ kg/mm²

For the purpose of comparison we also give the surge-load strength of side fillets:

Side-fillets { Gas-fusion welds fig. 11 $\rho_U = 14$ kg/mm²
Electric arc welds fig. 11 $\rho_U = 8{,}5$ kg/mm².

The gradual transition of weld metal to parent metal proved very favourable for gas-fusion welding. In par. 5c it will be shown that higher strength values are obtainable for arc-weldings if only the weld is given the same shape. From many trials with end fillet welds we learned that the fatigue-strength drops if at A in fig. 19 an inroad of weld metal with uneven surface (grooves) has taken place.

It is necessary to examine front fillet welds very carefully (magnifying glass) if the joint belongs to a dynamically stressed structural part. If flaws are found careful rewelding a A must be demanded. (This measure can also be recommended for structural framework, see 5c). On account of a too abrupt transition between weld metal and plate and an uneven surface at A an unfavourable notching action can be developed. This not only occurs for the weld passing almost the whole width of a member in tension, but also for end-fillet welds ending in a point.

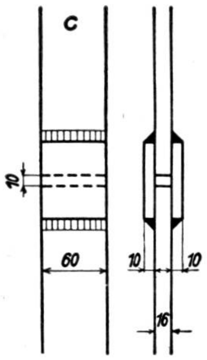


Fig. 17.

Fatigue tensile test with end fillet weld connections.

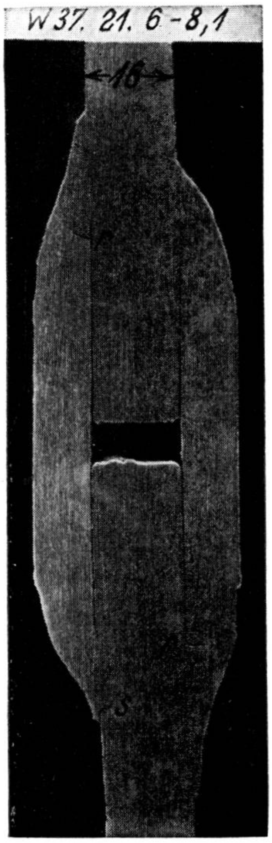


Fig. 18.

End fillet weld connections (gas fusion weld connections St 37) after frequently repeated tensile loading.

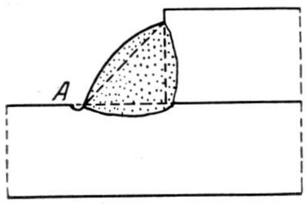


Fig. 19.

End-fillet welds placed right-angled to the direction of force for tensile members, strongly dynamically stressed, should be avoided (see 5c). Such caution is not necessary in building framework but also here grooves or notches as at A in fig. 19, should not be allowed.

The reduction of the fatigue-strength due to the employment of end-fillet welds right-angled to the flow of forces in tensile members, for instance in bridge building, indicates the advisability that web-stiffeners should not be welded together with the flange acting in tension. But a good fit between web stiffeners and the tensile flange is wanted, therefore a well-fitting distance piece usually forms the intermediary between flange and stiffener. These packing pieces are usually fixed with a spot weld to the stiffeners. The inner corners of the stiffeners between flanges and web plate are chamfered to prevent damage to the main welds whilst welding-on of the stiffeners. The gaps allow also for a good examination of the main weld. The stiffeners can be welded to the compressive boom of the girder. [(Distance pieces between web stiffener and tension chord are also advisable, for high-webbed girders in building frame constructions and other structural engineering.) The welding-on of stiffeners to the web plate causes shrinkage in the fillet welds, thus preventing a tight fit of the stiffener between the flanges, particularly for high-webbed girders. Through the employment of distance pieces the tight fitting of the stiffeners can be re-established.]

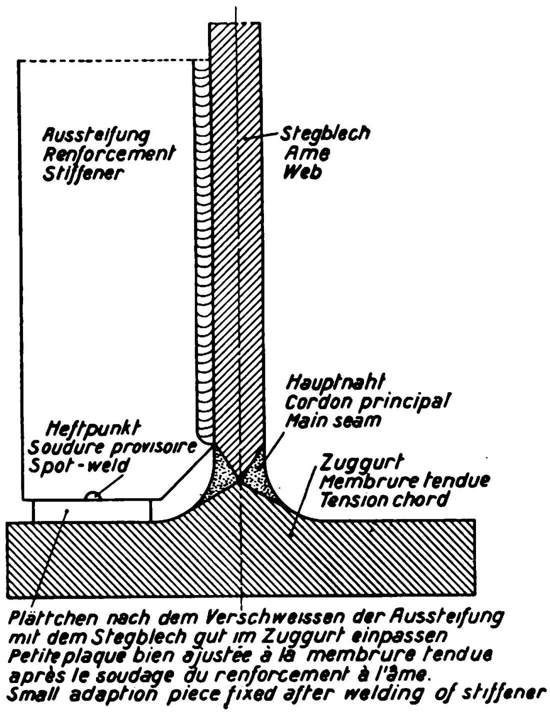


Fig. 20.

c) According to fatigue strength tests carried out in the Government test house in Dahlem with end-fillet welds the shape of the fillet is instrumental for the strength. The fatigue strength of end-fillet welds with the face under 45^0, welded with bare, unprotected welding-wires was found to be 10,8 kg/mm^2 for 2 kg/mm^2 pre-stressing and 2×10^6 loading repetitions. The fractures due to fatigue went through the plate at the edge of the weld.

Even by using welding-wires (electrodes) of higher ductility, up to a standard equal in quality to all mechanical properties of the parent metal, no better results are attainable if the tests are based on the same geometrical form of cross section (equal sides with bulging edges) of the fillet, as for the tests mentioned. The use of protected welding-wires (electrodes) for electric-arc welding is recommendable, producing as a rule a somewhat higher ductility of the weld, provided the particular properties of such electrodes allow the establishment of a more suitable cross section, especially a gradual transition of fillet surface to the parent metal.

In spite of all these difficulties the end-fillet welds must be regarded as an important structural element of high values particularly if only a suitable shape of the weld is chosen. The application of such welds can be without joining-up with side-fillet welds, but better still in connection with butt-welds.

At the outset of welding full end-fillet welds were regarded as better, but the results of fatigue strength tests have changed this idea completely. For all dynamically stressed fillet welds the cross sections shown above are better, particularly the concave section c in fig. 21.

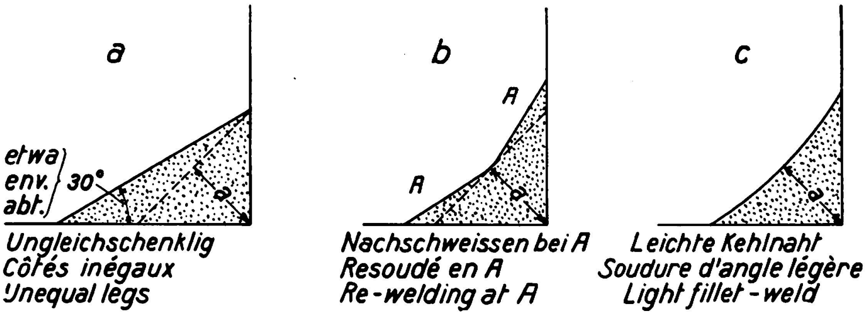


Fig. 21.

Suitable shapes of end fillet welds for dynamically stressed structural parts.

According to Table 2 of the joint report Dahlem-Dresden the following values for fatigue strength of fillets with an angle less than 45^0 were found.

		ρ_u	ρ_o	ρ_U
End-fillets	IV St. 37 Arc-welding	2,0	12,5	11,0 kg/mm²
	G IV St. 37 Gas-fusion-welding . .	2,0	10,8	9,3 kg/mm²

unprotected electrodes.

The tests carried out in Dahlem coincide with those of Stuttgart mentioned under section 5b as regards gas-fusion welding. Comparing the value found in Stuttgart of $\rho_U = 7\ \text{kg/mm}^2$ for an arc welded end fillet under 45^0 with the value found in Dahlem of $\rho_U = 9{,}3\ \text{kg/mm}^2$ it is obvious that the improvement is entirely due to the less steep face of the fillet. It does not seem improbable that by the use of protected electrodes or electrodes with core, an improvement in the transition of weld metal to parent metal can be established which has given for arc-welding increased values of the surge-load strength.

d) The surge-load strength for end-fillet welds shown in Fig. 22 (cross joint) could be considerably increased simply by tapering (chamfering) the plates. In these tests arc-welding was applied to plates of steel 52 using special electrodes. The results are as under

	Type	a	b	c
Static tensile test	σ_B	48,2	56,7	58,2 kg/mm²
Surge-load strength	ρ_U	9,5[11]	11	15 kg/mm²

The flow of forces is most favourable for the connections shown in fig. 22 and

[11] According to joint report Dahlem-Dresden see footnote page 13 table 2, G. II. E. (St. 37). The tensile fatigue strength with a pre-stressing of 2 kg/mm² was 10,5 kg/mm², corresponding to a surge-load strength $\rho_U = 9{,}5\ \text{kg/mm}^2$, the coincidence is evident.

it may be mentioned that on account of tapering the plates improved results for static tests are found as well (structural framework).

Mr. *Doernen* made use of these improvements particularly for welding together flange and web plates. He finally arranged for having flange plates of a special section (Rolling Mill Peine) manufactured enabling him in this way to replace the two fillet welds by an x-shaped butt-weld. See Fig. 23.

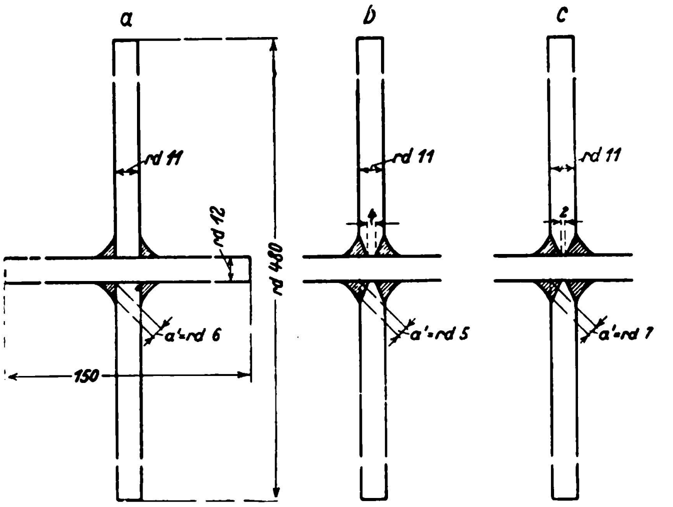


Fig. 22.

Welded cruciform connections (arc-welding).

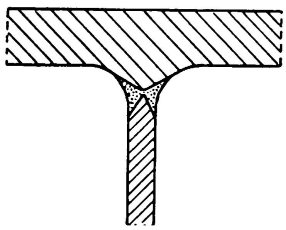


Fig. 23.

6) *Combination of butt and fillet-welds.*

The test house in Stuttgart studied the question of whether the fatigue strength of butt-welds could be increased by covering the joint with fish plates. Fish plates in themselves represent a reinforcement but the fillet-welds of these fish plates cause stress waves at various places, which depend on the shape of the welds and the ratio of the thicknesses of the fish plates to those of the main plates. In illustration 24 is shown that a butt-weld failed for a static tensile stress of $\sigma_B = 30{,}4$ kg/mm^2 but stood a tensile test of $\sigma_B = 38{,}4$ kg/mm^2 if the butt-weld was reinforced with welded-on fish plates. In this case it was the plate which fractured, showing the usual constriction, and not the weld.

In structural engineering full use can be made of the cross section of welded members by reinforcing the joint with fish plates. Illustration 24 also shows two fatigue tests for which the surge-load strength in case of a mere butt-weld was $\sigma_U = 9$ kg/mm^2 and for the same kind of joint but reinforced with fish plates $\sigma_U = 12$ kg/mm^2. The butt-weld with $\sigma_U = 9$ kg/mm^2 was not of good workmanship, therefore the reinforcement with fish plates was a considerable improvement, but still not so much as if the butt-weld had been of a better nature. It is shown in fig. 25 how a carefully executed butt-weld with $\sigma_U = 13$ kg/mm^2 was impaired by fish plates ($\sigma_U = 10$ kg/mm^2). The plate fractured at the end of the fish plate along the filled weld. The butt-weld connection in Fig. 26 where the root was not rewelded, gave a surge-load strength of $\sigma_U = 12$ kg/mm^2 which could be increased to $\sigma_U = 18$ kg/mm^2, by welding on fish plates machined as shown in fig. 26. The same value can be obtained with butt-welds, but with rewelded roots.

This experience will be made use of, for instance, in the case of joining flange plates of girders where the conditions of access do not permit rewelding of the root and therefore the soundness of the butt-weld remains doubtful. These tests

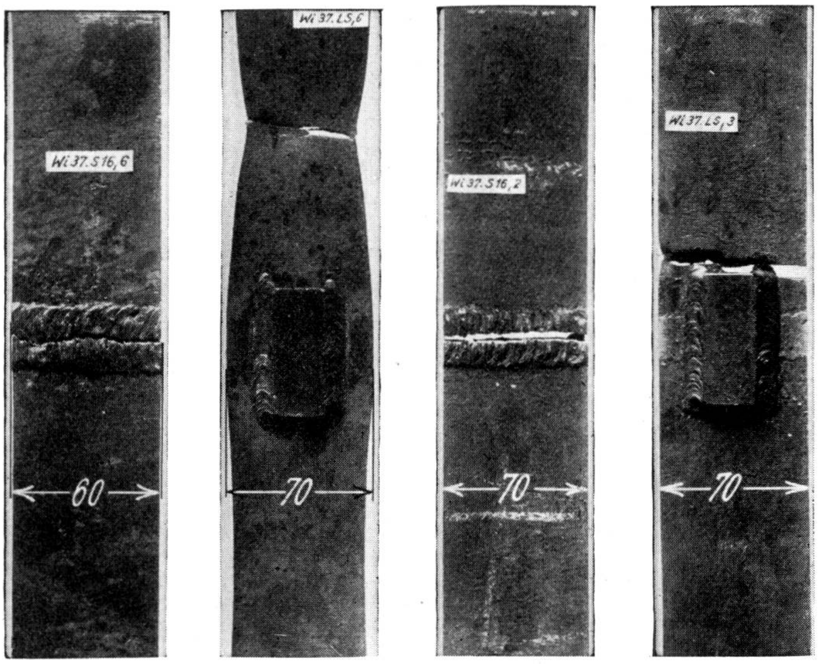


Fig. 24.

Static tensile tests σ_B 30.4 38.4 kg/mm²

Fatigue tensile tests. σ_U 9 12 kg/mm²

Butt welds with and without cover plates, St. 37, arc-welding,

prove also that it is advisable to taper the ends of flange plates (fig. 27) of dynamically stressed structures to establish a gradual flow of forces. For structural framework such arrangements are not absolutely necessary.

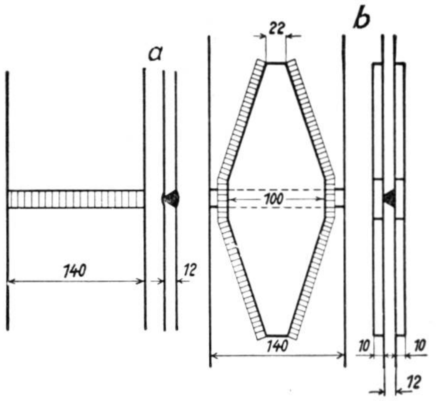


Fig. 25.

σ_U 13 10 kg/mm²

Fatigue tensile tests with arc-welds, St. 37, rewelded root of weld.

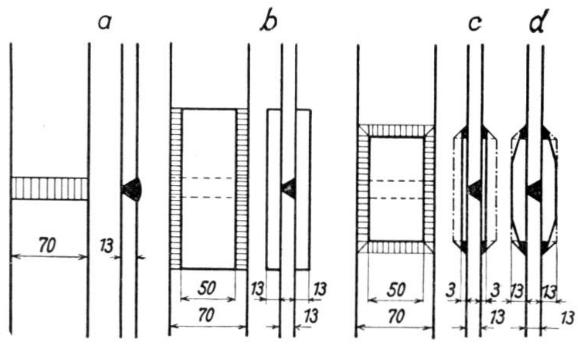


Fig. 26.

σ_U 12 13 17 18 kg/mm²

Fatigue tensile tests with gas-fusion welds St. 37. Root of weld not rewelded.

7) *Summary of fatigue test results*[12].

a) Welded connections merely statically stressed gave tensile resistances corresponding to the strength of the parent metal (σ = 37 to 42 kg/mm²). The tests showed the usual constrictions.

b) Welded connections of the same nature as above tested in pulsator-machines or swing bridges with two million loading repetitions reached a surge-load strength of only:

$$\sigma_U = 13 \text{ to } 18 \text{ kg/mm}^2 \text{ for butt-welds}$$
$$\sigma_U = 6{,}5 \text{ to } 10{,}3 \text{ kg/mm}^2 \text{ for end-fillet welds}$$
$$\sigma_U = 8 \text{ to } 12 \text{ kg/mm}^2 \text{ for side-fillet welds.}$$

The fractures had the usual characteristics for fatigue tests. The butt-welds have proved considerably superior to fillet-welds.

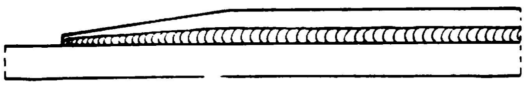


Fig. 27.

c) The fractures due to fatigue tests occured mostly in the parent metal and started from tiny notches in the surface at the transition between weld metal and plate (notching action).

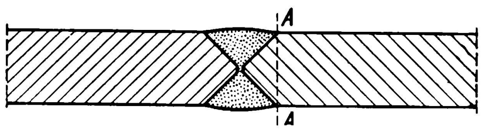


Fig. 28.

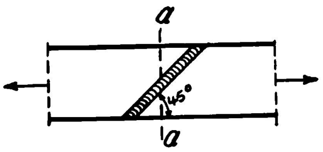


Fig. 29.
Butt weld under 45° of best workmanship.

d) For butt-welds, not rewelded at the root, the surge-load strength decreases to about 0,7 of the surge-load strength of butt-welds with rewelded roots.

e) If carefully executed butt-welds under an angle of 45° are introduced in tensile members, the surge-load strength increases for steel 37 from σ_U = 18 kg/mm² up to σ_U = 22 kg/mm². The specimens fractured along line aa, Fig. 29

f) For a particular test with a butt-weld connection, reinforced with fish plates and fixed by fillet-welds, the surge-load strength decreased from 13 to 10 kg/mm² (the butt-weld itself was not of excellent workmanship). The specimen broke at the beginning of the fillet-welds near the fish plate.

g) For fillet welds the fatigue strength dropped considerably if the weld metal did not properly enter into the root of the fillet.

h) Contrary to previous ideas light end fillet welds with a gradual transition between weld-metal and plate proved superior to end-fillet welds of full section.

i) At all places having end-fillet welds or where structural parts are fixed by side-fillet welds to other structural parts, in fact everywhere there where

[12] See *Kommerell:* Erläuterungen zu den Vorschriften für geschweißte Stahlbauten, 4. Auflage, II. Teil Vollwandige Eisenbahnbrücken.

side-fillet welds start or end, the permissible stresses in the structure itself must be reduced to

$$\sigma = \alpha \sigma_{zul} \quad (zul = \text{permissible})$$

The coefficient α is dependent on the ratio $\frac{\min M}{\max M}$ (see Table 2 V line 14 to 17 page 32 Preliminary Publication).

k) A higher value of surge-load strength can be obtained at all places where fillet-welds start or end provided it is possible to form a gradual transition of the weld metal to the parent metal.

l) The cross sectional shape of a weld, particularly the transition of weld metal to parent metal, is more instrumental as regards strength than the nature of the electrodes used for welding.

m) The results of fatigue tests obtained with swing bridges agree essentially with those obtained with pulsator machines.

n) A distinct difference in fatigue strength between structural elements of steel 37 from those of steel 52 was not found. The properties of high grade steel only enter into account after pre-stressing.

o) Fatigue tests have shown that the surge-load strength is sometimes very low, compared with the tensile strength σ_B obtained with the same specimens in static tests. It is therefore necessary also to test the electrodes (welding wires) also for fatigue strength.

p) Fatigue tests carried out after the publication of the report of the Board of Administrators have shown that the surge-load strength of fillet welds stressed longitudinally is the same as for butt-welds $\sigma_U = 16—18$ kg/mm^2.

q) Originally, fillet welds were considered better and more reliable than butt-welds, particularly for structural parts dynamically stressed in tension, but fatigue tests have proved the superiority of properly executed butt-welds. For butt-welds the flow of forces is more natural, while for fillet welds the forces very often have to undergo a distinct deviation from the original direction and peak stresses are created simply by the sudden change in cross section. The importance of an undisturbed flow of forces has shown itself chiefly for welded connections dynamically stressed. As the permissible stresses for butt-welds can be higher than those for fillet welds and, as will be shown later, influence accordingly the dimensions of pieces to be welded, economical reasons will therefore demand, wherever possible, to choose butt-welds for dynamically stressed structural parts. It embodies also the advantage that for butt-welds the soundness of workmanship and material can be examined more easily than for fillet welds (X-Rays).

r) Formerly when calculating welded bridges it was regarded as sufficient to allow only for amply sized welds, to secure in this way a faultless execution of the whole structure. From the results of fatigue tests however we learned to pay equal attention to the parts to be welded, since testing has shown that often the specimens fractured and not the weld. Usually fracture occurs at the transition from parent metal to weld metal, therefore the permissible stresses in welded members shall not be more than those for the weld itself.

s) Generally speaking, arc-welding and gas-fusion welding can be considered as equal. Even if the lower values of surge-load strength for gas-welded butt-joints are higher, for both types of welding the same maximum values ($\sigma_U = 18$ kg/mm²) were attained, and the same is true for side-fillet welds. For both systems of welding the fractures have their origin at the ends of side-fillet welds. Gas-welded end-fillet welds only gave better values on account of the more suitable cross sectional shape of the weld, having a gradual transition between weld metal and plate. Equally good results were obtained with arc welding if the section of the weld was given the same shape as above.

D. The permissible stresses $\sigma_{D\,zul}$ in respect to fatigue strength.

1) *General.*

A clear and straightforward mode of construction is only possible if structural parts acting in tension or bending can be butt-welded without the necessity of providing special cover or fish plates. The butt-welds must obviously be at such places where the permissible stresses for the structural element itself are not overstepped.

2) *The γ-Method*[13].

The regulations of the German State Railways[14] (BE) for calculating railway bridges in steel say that all parts of a superstructure should be designed to have the same factor of safety. This rule can be established in the simplest and clearest manner if all stresses are brought in relation to the permissible stress σ_{zul} which represents the permissible bending stress for unjointed structural parts. (For the calculation of compressive members, for instance, the formula

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

should be used wherein ω is a coefficient relating to the buckling conditions. The compressive force S for centric action multiplied by the coefficient ω allows, in respect to the permissible stress, a member in compression to be treated in the same way as a member in tension.) A similar procedure (γ-method) has been introduced by the German State Railways for riveted as well as welded bridges. This method allows of the consideration in calculation of the fatigue strength σ_D of the material for all such parts as are subject to alternating or surging stresses.

If $\sigma_{D\,zul}$ indicates the permissible stress under consideration of the fatigue strength (generally less than σ_{zul}) the calculation of plate girders is based on the following regulation

$$\sigma_{D\,zul} = \frac{\max M}{W} = \frac{\sigma_{zul}}{\gamma} \tag{2}$$

hence

$$\sigma = \gamma \cdot \frac{\max M}{W} = \sigma_{zul}. \tag{3}$$

[13] See *Kommerell:* Verfahren zur Berechnung von Fachwerkstäben und auf Biegung beanspruchten Trägern bei wechselnder Belastung. „Bautechnik 1933", page 114.

[14] Berlin 1934, to obtain from the Reichsbahn-Zentralamt Berlin, Halle'sches Ufer.

Comparing the term $\gamma \cdot \frac{\max M}{W}$ with the value σ_{zul} we see that γ represents a figure ($\geqq 1$) with by the maximum bending moment requires to be multiplied to enable the calculation of the girders to be carried out in such a way as if the girder is subject only to a bending moment max M produced by a constant load (as for instance in building construction).

Under consideration of an impact coefficient φ, the terms $\frac{\min M}{\max M}$ represent the extreme values of bending moments (the most unfavourable limits produced by the passing of a train). The term min M stands for the numerically smallest and max M for the numerically highest value of the bending moments. For girders with the loading remaining permanently unchangeable we have

$$\sigma = \frac{\max M}{W} \leqq \sigma_{zul} \tag{4}$$

For bridges, however, where rolling loads create alternating stresses (stress limits of reverse sense) or surging stresses (stress limits of equal sense), not only the highest bending moment max M but also a portion of the smallest bending moment requires to be considered. The extent of these influences is laid down in the following formula and ruled by the coefficients a and b, which require to be defined specially.

$$\sigma = \frac{a \cdot M\max + b \cdot \min Max}{W} = \left(a + b \cdot \frac{\min M}{\max M}\right) \cdot \frac{\max M}{W} \leqq \sigma_{zul}. \tag{5}$$

The bracket represents the value γ in formula (3),

hence $$\gamma = a + b \cdot \frac{\min M}{\max M} \quad \text{is} \tag{6}$$

a linear function of $\frac{\min M}{\max M}$. For riveted bridges γ has the value

$$\gamma = 1 - 0{,}3 \cdot \frac{\min M}{\max M} \tag{7}$$

In my report to the 1st Congress in Paris on page 332 I showed that formerly the calculation for welded joints on bridges was based on the following formula

$$M = \max M \cdot \frac{1}{2} (\max M - \min M) = \max M \left(1{,}5 - 0{,}5 \frac{\min M}{\max M}\right), \tag{8}$$

hence

$$\gamma = 1{,}5 - 0{,}5 \frac{\min M}{\max M}. \tag{9}$$

On table 2 page 6 in the report to the Congress in Paris it is shown that the fatigue strength varies with the type of weld and is dependable also on the workmanship. The resistance to alternating effects varies between 4,2 and 13,0 kg/mm² and the surgeload strength between 8,0 and 22,0 kg/mm².

In the report of the Board of Administrators on page 46 par. 16 and 17, I originally recommended also for fillet welds, which should not be stressed as high as through members, the employment of special values for γ. After further consideration I gave up this idea as otherwise the large number of other cases would have necessitated the calculation of special γ-values, provided that in such cases the permissible stresses should be less than for through-members. A large number of such γ-values would have rendered the regulations very complicated and intricate. Even the attempt to make the regulations as simple as possible by introducing values for γ suited for the most unfavourable cases only, with a single reducing coefficient $\alpha = 0{,}65$ proved unsuccessful as the values for γ would have become too high in this way. The result would have been that statically indeterminate welded girders would no longer have been competitive compared with riveted constructions. The difficulty was solved by introducing variable values for γ (form-characteristics) see table 2 V and 3 V, page 32 of the report to the Congress in Paris. If under consideration of the γ-values unwelded (or through-) members can be permissibly stressed up to $\left\{\begin{array}{ll} 1400\ \mathrm{kg/cm^2} & \text{for st. 37} \\ 2100\ \mathrm{kg/cm^2} & \text{for st. 52} \end{array}\right\}$ then the permissible stress should be, for cases where such high stressing values are not permissible, $\sigma'' = \alpha \cdot \sigma_{zul}$. The values α can be taken from the table 2 V and 3 V. With the intention of bringing all these values in harmony with σ_{zul}, the following term was introduced

$$\frac{\sigma''}{\alpha} \leqq \sigma_{zul}$$

leading to the formula

$$\sigma_1 = \frac{\gamma}{\alpha} \cdot \frac{\max M_1}{W_n} \leqq \sigma_{zul} \qquad (5\,V)$$

3) *Diagrams of permissible stress limitations.*

It is clear that the permissible stresses $\sigma_{W\,zul}$, $\sigma_{U\,zul}$ should not be taken as high as the values of fatigue strength as produced by $2 \cdot 10^6$ loading repetitions in pulsator machines or swing bridges, even if the oscillating loadings of strength calculations are only attained rarely. It is possible that in the interior of the material and in welded connections irregularities exist which cannot be detected by the most thorough method of examination. The fixing of the interval between the fatigue strength in pulsator machines with $2 \cdot 10^6$ loading repetitions and the permissible stresses of welded connections was done by the Working Committee, who accepted my advice regarding the excellent experiences made with riveted railway bridges. According to these experiences the permissible stress for steel 37 is $\sigma_{zul} = 1400\ \mathrm{kg/cm^2}$ for main influences only and laid down in the BE (Basis of calculation for railway bridges in steel). Including wind and other additional forces the permissible stress can be increased up to

$$\sigma_{zul} = 1600\ \mathrm{kg/cm^2}.$$

Wind and other additional forces do not occur with the passing of every train, they have much more the significance of pre-loading (dead weight). The taking into account of wind and other additional forces resembles an increase in stresses due to dead weight of about $200\ \mathrm{kg/cm^2}$. The amplitude σ_w remains identical within the respective range of stresses, from which it follows that the fatigue strength under consideration of wind and additional forces also increases accordingly by about 200 kg/cm². It suffices therefore to make clear the conditions resulting out of all main forces only. For the purpose of comparison, the fatigue strength test results of riveted specimens have been studied as well. The surge-load strength of riveted connections (of similar dimensions as for welded connections) was found to be $\sigma_{U\,zul} = 15$ kg/mm² (see *Woehler* line, page 16 Part 34, report of the Board of Administrators). Such values or less were obtained fairly frequently with pulsator machines, especially if a coat of red lead-oxyde was covering the contact areas of lap-jointed plates. The interval between surge-load strength and permissible stress is

$$\sigma - \sigma_{zul} = 15 - 14 = 1 \text{ kg/mm}^2.$$

Properly executed welded connections are equal and often superior to riveted connections, no reason therefore exists for subjecting welded bridges to less favourable regulations than for riveted bridges. The Working Committee decided to fix the σ_{zul}-values at 1 kg/mm² less than the fatigue strength values for $2 \cdot 10^6$ loading repetitions. This procedure makes it unnecessary to study wether the rapid sequence of loading repetitions in pulsator machines allows conclusions to be drawn in respect to the stressing of railway bridges, where, anyhow, the change of loading is very slow.

The values for the permissible stress σ_{zul} can therefore be throughout 1 kg/mm² less than the fatigue strength values for $2 \cdot 10^6$ loading repetitions.

The fundamental elements for calculating welded plate girders for railway bridges are the diagrams of permissible stress limitations which are shown in fig. IV for steel 37 and 2 V for steel 52 which have been laid down finally by the Working Committee. As regards details I wish to refer to my "explanations" Part II, page 30 etc. The mode of illustrating is in conformity with figures 1 and 2, on page 3 and 4.

4) *Explanations concerning the various $\sigma_{D\,zul}$-lines.*

The fatigue strength values σ_D given in the report of the Board of Administrators form the basis (see table 2, page 6)

α) Steel 37

1) Lines Ia and Ib apply for unjointed structural parts (through members) in tension or compression.

According to table 2, row 3 line 1 for butt-welds of first quality (rewelded root and gradual transition between weld metal and plate) $\sigma_{W\,zul} = 11$ kg/mm² and for $\min M = -\max M$ a stress $\sigma_{W\,zul} = 11 - 1 = 10$ kg/mm² would be permissible. Based on new tests the representatives of test houses in the Working Committee considered it advisable to fix the value $\sigma_{W\,zul}$ as under

$$\sigma^{Ia,\,Ib}_{W\,zul} = \pm\, 10{,}8 \text{ kg/mm}^2$$

Table 2 for the values σ_U shows for such welds a stress of $\sigma_U = 18$ kg/mm² according to which

$$\sigma_{U\,zul} = 18 - 1 = 17 \text{ kg/mm}^2 \text{ is possible}$$

The figures are in kg/mm².

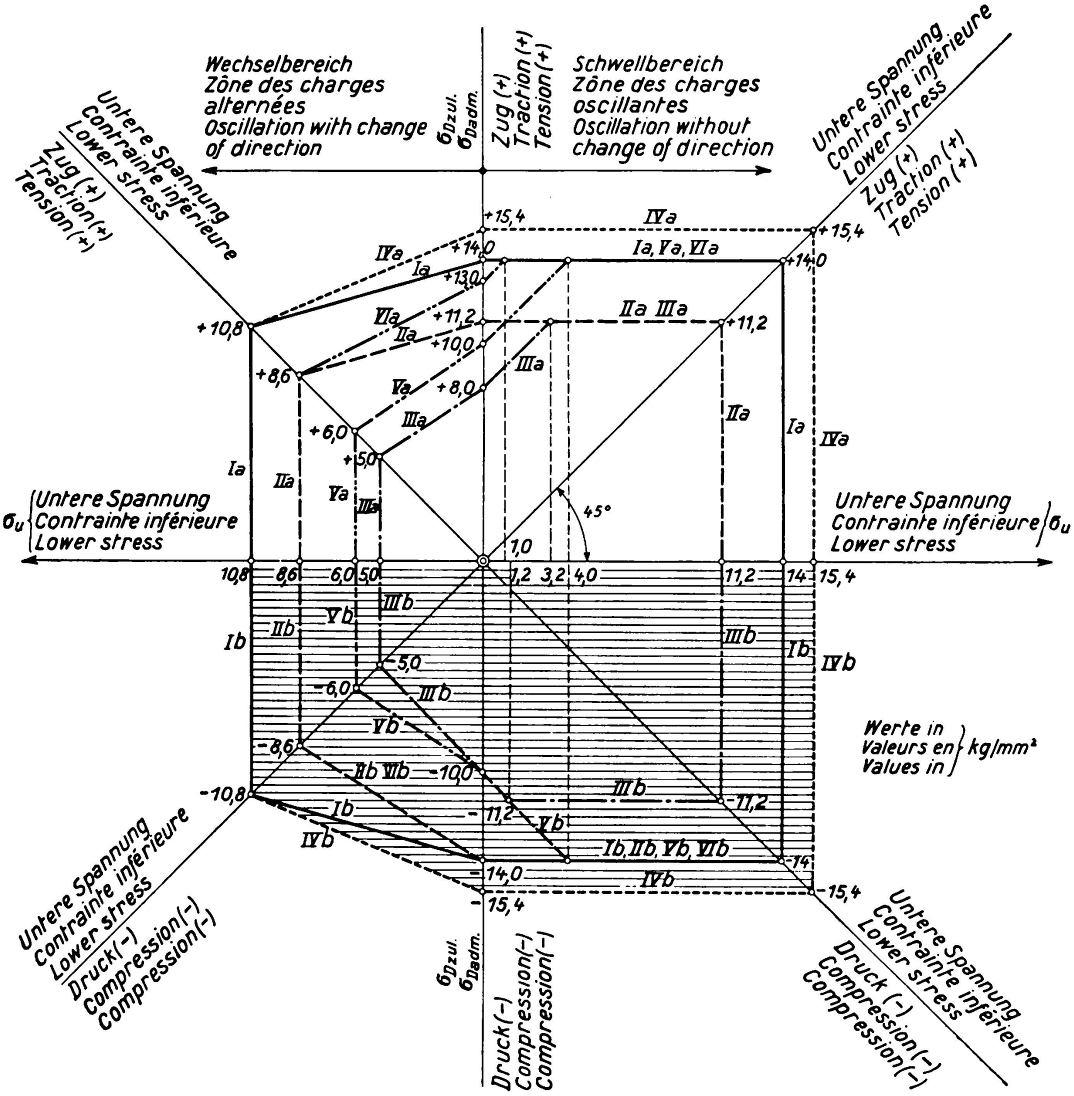


Fig. 1V.

Diagram of permissible stresses σ_{Dzul} for welded bridges in St. 37. The figures express stresses in kg/mm².

Ia, Ib	Unjointed members (through-members) in tension or compression.
IIa	Jointed members in tension for butt-welds and the vicinity of butt-welds provided the roots are rewelded and the welds are tooled or machined.
IIb	Same as for IIa but in compression.
IIIa, IIIb	Same as for IIa and IIb in cases where the root cannot be rewelded.
IVa, IVb	Permissible main stresses according to formula

$$\sigma = \frac{\sigma_1}{2} + \frac{1}{2}\sqrt{\sigma_1^2 + 4\tau_1^2}$$

Va, Vb	Structural members in the vicinity of end-fillet welds. Untreated end-fillet weld transition and untreated ends of side fillet welds.
VIa, VIb	Same as for Va and Vb with careful machining of end-fillet weld transitions and ends of side-fillet welds.

The figures are in kg/mm².

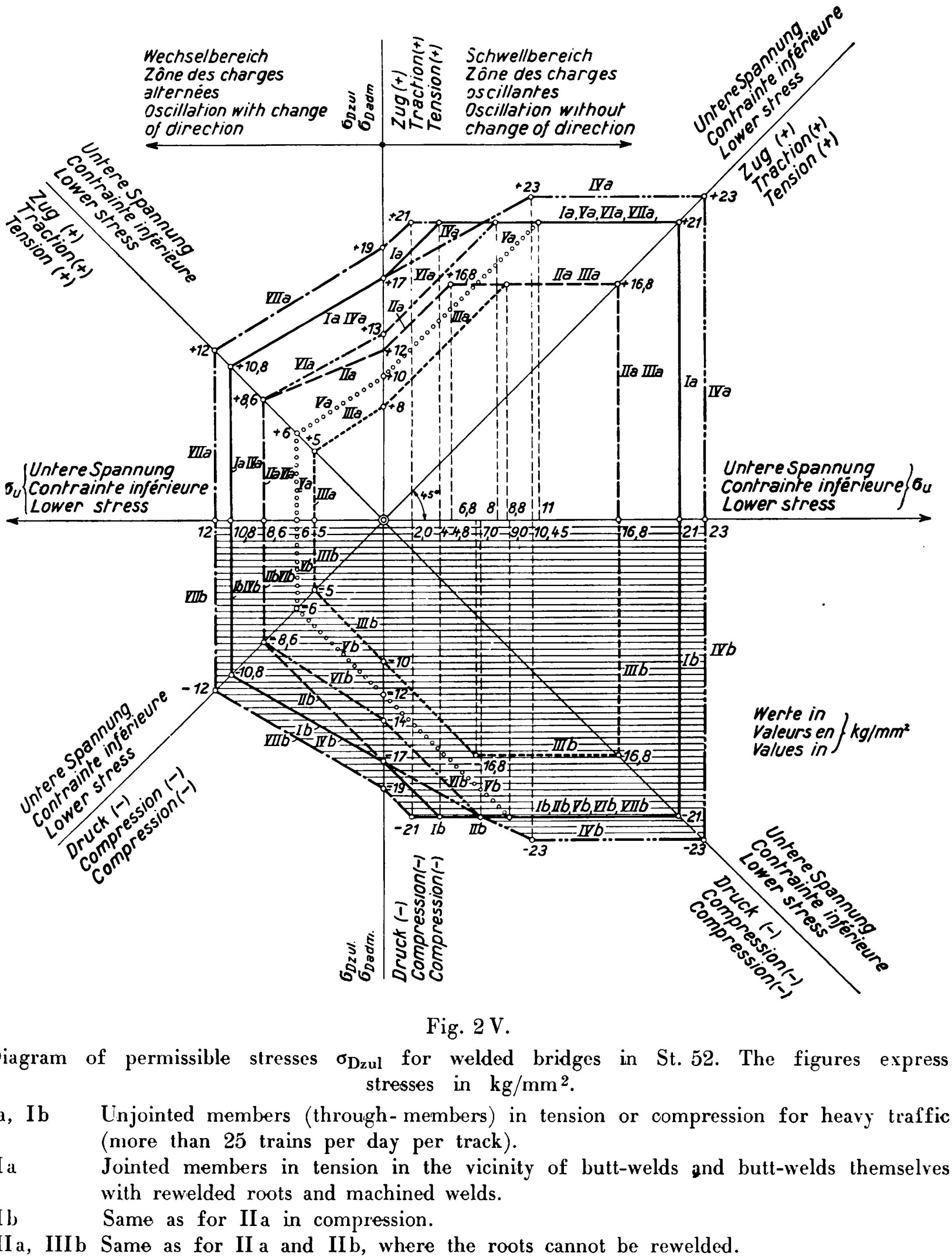


Fig. 2 V.

Diagram of permissible stresses σ_{Dzul} for welded bridges in St. 52. The figures express stresses in kg/mm².

Ia, Ib Unjointed members (through-members) in tension or compression for heavy traffic (more than 25 trains per day per track).

IIa Jointed members in tension in the vicinity of butt-welds and butt-welds themselves with rewelded roots and machined welds.

IIb Same as for IIa in compression.

IIIa, IIIb Same as for IIa and IIb, where the roots cannot be rewelded.

IVa, IVb Permissible main stresses according to formula

$$\sigma = \frac{\sigma_1}{2} + \frac{1}{2} \sqrt{\sigma_1^2 + 4\tau_1^2}$$

Va, Vb Structural members in the vicinity of end-fillet welds and at the beginning of side-fillet welds. Untreated end-fillet weld transitions and ends of side-fillet welds.

VIa, VIb Same as for Va and Vb with careful machining of end-fillet weld transitions and ends of side-fillet welds.

VIIa, VIIb Same as for Ia and Ib, for light traffic (up to 25 trains per day per track).

But it was not considered advisable to go beyond

$$\sigma_{U\,zul}^{Ia,\,Ib} = \pm 14\,kg/mm^2$$

and the Working Committee decided that for min M = 0 this value shall apply for the tensile as well as for the compressive zone. The lines Ia and Ib for welded bridges in steel 37 are in this case the same as for for riveted bridges. The lines Ia and Ib do not apply as originally intended for butt-welds of best workmanship, but only for unjointed through members. On account of the various modes of execution for butt-welds a number of special lines were laid down (see Par. 2 and 3).

2) The line IIa for jointed members in tension, in the vicinity of butt-welds and butt-welds themselves, with rewelded roots and machined welds.

Although the fatigue strength figures in table 2 are derived from tensile tests, the Working Committee decided for the tensile zone to reduce the values σ_{zul} to 0,8 of the values of line Ia, thus allowing in return members in tension to be welded without requiring the welded joints to be specially covered with cover or fish plates. To increase the factor of safety, butt-welds under 45^0 with rewelded roots and gradual transition between weld metal and plate, should be arranged in tensile members, and hence we have

$$\sigma_{W\,zul}^{IIa,\,IIb} = \pm 10{,}8 \cdot 0{,}8 = \sim \pm 11{,}2\,kg/mm^2$$
$$\sigma_{U\,zul}^{IIa} = 14 \cdot 0{,}8 = 11{,}2\,kg/mm^2$$

For the zone of compression the line IIb

$$\sigma_{U\,zul}^{IIa} = -\ 14{,}0\,kg/mm^2$$

was laid down for unjointed, through-members (line Ib).

3) For line IIIa the same applies as for IIa provided the roots cannot be rewelded. The Working Committee fixed the stressing values for this rare case to

$$\sigma_{W\,zul}^{IIb} = 5{,}0\,kg/mm^2$$
$$\sigma_{U\,zul} = 8{,}0\,kg/mm^2$$

In the tensile as well as in the compressive zone the lines $\sigma_{D\,zul}$ shall be under an angle of 45^0, as it is permissible to assume equal amplitudes of stresses for the range of surging forces. According to this, the line IIIa reaches a stress of 11,2 kg/mm² with a lower stress of 11,2—8 = 3,2 kg/mm². For line IIIb we have

$$\sigma_{U\,d\,zul}^{IIIb} = -\ 10\,kg/mm^2$$

The line IIIb passes, with a lower stress value of —(11, 2—10,0) = —1,2 kg/mm², through the point — 11,2 kg/mm².

4) The lines IVa and IVb apply for permissible main stresses based on the formula

$$\sigma = \frac{\sigma_1}{2} + \frac{1}{2}\sqrt{\sigma_1^2 + 4\,\tau_1^2}$$

In case of min M = — max M the same values

$$\sigma_{W\,zul}^{IVa,\,IVb} = \pm 10{,}8 \text{ kg/mm}^2$$

were stipulated as for the lines Ia and Ib. But as for steel 37, if min M = 0, the fatigue strength is not made full use of, the Working Committee decided to allow

$$\sigma_{U\,zul}^{IVa,\,IVb} = 15{,}4 \text{ kg/mm}^2 \left(14 + \frac{14}{10}\right).$$

The report of the Board of Administrators concerns only tensile tests where the specimen, subject to fatigue tests, fractured as a rule at A or B, at the beginning of the side-fillet welds (change of cross section see fig. 30).

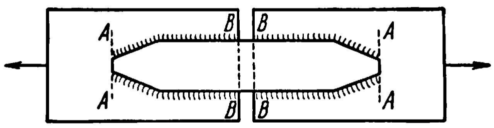


Fig. 30.

As the welds connecting web plate and flange consist as a rule of through fillets or butt-welds, I found it necessary to arrange for fatigue tests for such through-welds as were carried out in the test house of the Central State Railway Purchase Department in Wittenberge. For these tests in pulsator machines through fillet and butt-weld specimens were used according to illu-

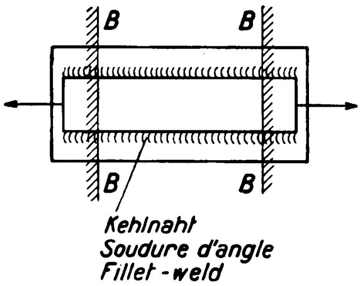


Fig. 31.

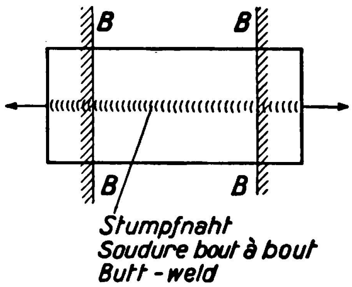


Fig. 32.

strations 31 and 32. The results of these tests proved, as expected, that such through fillet and butt-welds can be stressed equally as high as butt-welds right angled to the direction of the force. In fact surgeload strengths of

$$\sigma_U = 18 \text{ kg/mm}^2$$

were easily attained. (For the purpose of comparison we examined as shown in illustration 3 specimens with interrupted fillet-welds giving surge-load strengths of smaller values on account of the notching influences.)

5) Lines Va, Vb for structural parts in the vicinity of end-fillet welds and at the beginning of side fillet welds, the ends of these welds not machined.

The Working Committee at its meeting in Goslar had originally in mind to consider only machined fillet welds. The Committee was of the opinion that on account of the already low values of fatigue strength for such welds only machined fillet-welds (machined ends of side fillet welds) should be considered. It was proposed therefore according to table 2, row 6 to allow only a permissible stress of

$$\sigma'^{a,\,b}_{W\,zul} = \pm (6{,}3 - 1{,}0) = \pm 5{,}3\ \mathrm{kg/mm^2}$$

But finally the Committee increased this value to $\pm$ 6,0 kg/mm² and with it

$$\sigma'^{a,\,b}_{U\,zul} = \pm 10\ \mathrm{kg/mm^2}.$$

In the zone of surging loads the line $\sigma_{D\,zul}$ shall run under 45° to pass the values $\pm$ 14 kg/mm² with a lower stress of 4 kg/mm², this applies for the zone of tensile as of well as compressive forces.

In the meantime *Dr. Doernen* carried out fatigue tests with fillet welds with a gradual transition to the parent metal and flange plates with chamfered edges.

The Working Committee, meeting this time in Friedrichshafen, considered it worth while to accept the results found by *Dr. Doernen* for untreated fillet and butt-welds and stipulated the following values

$$\sigma^{Va,\,Vb}_{W\,zul} = \pm\ 6\ \mathrm{kg/mm^2}$$
$$\sigma^{Va,\,Vb}_{U\,zul} = \pm 10\ \mathrm{kg/mm^2}$$
$$\sigma^{Va,\,Vb}_{D\,zul} = \pm 14\ \mathrm{kg/mm^2}$$

with a lower stress of 4 kg/mm²

6) Lines 6a and 6b for structural parts in the vicinity of end fillet-welds and at the beginning of side fillet-welds for machined ends of the welds.
Based on the results of a number of new tests it was decided at the meeting at Friedrichshafen to adopt the following permissible stresses

$$\sigma^{VIa,\,VIb}_{W\,zul} = \pm\ 8{,}6\ \mathrm{kg/mm^2}$$
$$\sigma^{VIa}_{U\,zul} = +13{,}0\ \mathrm{kg/mm^2}$$
$$\sigma^{VIb}_{U\,zul} = -\ 14{,}0\ \mathrm{kg/mm^2}$$

β) Steel 52.

According to the report of the Board of Administrators the fatigue strength tests have proved that the surge-load strengths for steel 52 (as well as the strengths for alternating effects) are only little higher than the corresponding values for steel 37.

1) Lines Ia, Ib for unjointed through-members in tension or compression (heavy traffic)

$$\sigma^{Ia,\,Ib}_{W\,zul} = \pm 10{,}8\ \mathrm{kg/mm^2}$$

According to table 2, line 1, the value σ_U = 18 kg/mm² was found for butt-welds with rewelded roots allowing a permissible stress of

$$\sigma^{Ia,\,Ib}_{U\,zul} = \pm (18 - 1) = 17\ \mathrm{kg/mm^2}$$

The lines $\sigma_{D\,zul}$ in the area of surging loads were again taken under 45^0 with the values

$$\sigma^{Ia,\,Ib}_{D\,zul} = \pm 21 \text{ kg/mm}^2$$

and a lower stress of 4 kg/mm².

2) Lines IIa and IIb for jointed members in tension or compression and machined butt-welds with rewelded roots.
For this case the following values have been laid down for tensile stresses

$$\sigma^{IIa}_{W\,zul} = 0{,}8 \cdot 10{,}8 = \sim + 8{,}6 \text{ kg/mm}^2 \text{ (as for st. 37)}$$
$$\sigma^{IIa}_{U\,zul} = + 12 \text{ kg/mm}^2$$
$$\sigma^{IIa}_{D\,zul} = + 0{,}8 \cdot 21 + 16{,}8 \text{ kg/mm}^2$$

with a lower stress of $+ 4{,}8$ kg/mm².

For compression the following values apply

$$\sigma^{IIb}_{W\,zul} = - 8{,}6 \text{ kg/mm}^2$$
$$\sigma^{IIb}_{U\,zul} = - 2 \cdot 8{,}6 = \sim - 17 \text{ kg/mm}^2$$
$$\sigma^{IIb}_{D\,zul} = - 21 \text{ kg/mm}^2$$

with a lower stress of $- 4$ kg/mm².

3) Lines VIIa and VIIb for unjointed through members in tension or compression for light traffic.

As for riveted bridges (see BE, table 17), distinction should be chawn for bridges in steel 52, between bridges for heavy traffic with more than 25 trains per day and such for light traffic with up to 25 trains per day per track), the Working Committee at its meeting in Friedrichshafen decided to adopt the following values for permissible stresses

$$\sigma^{VIIa,\,b}_{W\,zul} = \pm 12 \text{ kg/mm}^2$$
$$\sigma^{VIIa,\,b}_{U\,zul} = \pm 19 \text{ kg/mm}^2$$
$$\sigma^{VIIa,\,b}_{D\,zul} = \pm 21 \text{ kg/mm}^2$$

for a lower stress value of $\pm$ (21—19) = 2 kg/mm².

4) The remaining diagrams of stress limitations for steel 52 have been arranged according to the same rules as for steel 37. In this connection I refer herewith to my "Explanations" Part II, page 34 etc.

E. Calculation of sections.

1) *General.*

a) The following rules should be observed in calculating such structural parts as are subject to alternating or surging forces:

max M_1 indicates numerically the maximum and
min M_1 numerically the minimum bending moment

values out of dead weight and life load with an impact coefficient φ (for bridges in curves must be included also the centrifugal forces multiplied by the impact factor φ); positive bending moments receive the sign (+), negative bending moments (compressive forces) are marked (—).

If for instance

$$M_g = +200 \text{ tm}$$
$$\varphi M_p = +400 \text{ tm}$$

and

$$\varphi M_p = -600 \text{ tm}$$

hence it is $\max M_I = +200 + 400 = +600$ tm

$$\min M_I = +200 - 600 = -400 \text{ tm}$$

If $\max M_I$ and $\min M_I$ have reverse signs we speak of alternating efforts, but if $\max M_I$ and $\min M_I$ have the same signs we speak of surging efforts.

b) Provided it were not be necessary to distinguish for welded connections between the type and the position of welds, the influences of alternating or surging loads could be treated according to the γ-method (B E, Para. 36) as for riveted bridges in which case the formula

$$\sigma' = \frac{\gamma \max M_I}{W_n} \text{ could apply.} \qquad (4\,V)$$

In this formula γ indicates a coefficient depending on the influences of alternating or surging loads (fatigue stresses). With this coefficient, the numerical maximum value of bending moments (consisting of the bending moments due to dead weight, life load, centrifugal forces with impact coefficient φ) will be multiplied to enable the structural elements to be calculated in such a way as if no alternating or surging efforts existed.

c) The different values for fatigue strength for the various types of welded connections receive consideration through a coefficient α derived from the diagrams of stress limitations in fig. 1 V and 2 V and the tables 2 V and 3 V. Accordingly we have

$$\sigma_1 = \frac{\gamma \max M_I}{\alpha W_n} \leqq \sigma_{zul} \qquad (5\,V)$$

(1400 kg/cm² for st. 37, 2100 kg/cm² for st. 52.)

(Shear forces receive consideration by multiplying their values by a coefficient $\frac{\gamma}{\alpha}$ wherein for the γ-values the terms $\frac{\min M_I}{\max M_I}$ are replaced by $\frac{\min Q_I}{\max Q_I}$.)

d) The γ-values are derived from the diagrams of stress limitations in the case of $\alpha = 1$, which for instance exists for butt-welds of first quality for the range of surging forces in compression.

e) The γ-values are dependent on the ratio $\frac{\min M^I}{\max M_I}$ subject to the signs of $\min M_I$ and $\max M_I$.

The method of making use of the coefficient α has the advantage that everything is based on the same scale, namely σ_{zul}. This enables the designer to see where and how much the permissible stresses have to be reduced. This method has also an educative value as it automatically forces the designer to be economical, particularly in the case of small α values (for instance he will be induced to employ a different type of structural detail at places of beginning fillet welds). If the stresses σ_r are required to be known, without the consideration of the values γ and α, it is only necessary to multiply the value σ out of formula 5 V by $\frac{\alpha}{\gamma}$.

Example: $$\sigma = \frac{1,2}{0,65} \cdot \frac{\max M_I}{W} = 1380\,\text{kg/cm}^2$$

hence $$\sigma_r = 1380 \cdot \frac{0,65}{1,2} = 750\,\text{kg/cm}^2$$

2) *The coefficients* γ.

Based on fig. 33 we have

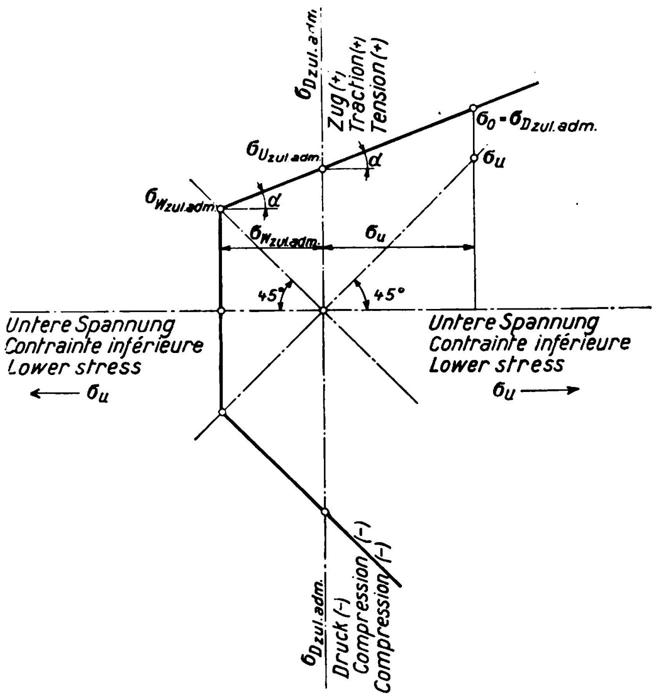


Fig. 33.

$$\operatorname{tg} \alpha = \frac{\sigma_{U\,zul} - \sigma_{W\,zul}}{\sigma_{W\,zul}} \quad \text{and} \tag{10}$$

for any point (σ_o, σ_u) exists the relation

$$\sigma_o - \sigma_{U\,zul} = \sigma_u \cdot \operatorname{tg} \alpha \tag{11}$$

hence $$\sigma_o - \sigma_u \cdot \operatorname{tg} \alpha \cdot \frac{\sigma_o}{\sigma_o} = \sigma_{U\,zul}$$

or $$\sigma_o = \frac{\sigma_{U\,zul}}{1 - \operatorname{tg} \alpha \cdot \dfrac{\sigma_u}{\sigma_o}} \tag{12}$$

therefore $$\sigma_u = \frac{\min M_I}{W} \tag{13}$$

$$\sigma_o = \sigma_{D\,zul} = \frac{\max M_I}{W} \tag{14}$$

hence $$\frac{\sigma_u}{\sigma_o} = \frac{\min M_I}{\max M_I} \tag{15}$$

with these terms we receive out of (12)

$$\sigma_o = \sigma_{D\,zul} = \frac{\sigma_{U\,zul}}{1 - \operatorname{tg}\alpha \cdot \dfrac{\min M_I}{\max M_I}}. \tag{16}$$

According to the B E σ_{zul} the permissible stress is $\left\{\begin{matrix} 1400\ \text{kg/cm}^2 \text{ for steel } 37 \\ 2100\ \text{kg/cm}^2 \text{ for steel } 52 \end{matrix}\right\}$.
If the values for $\sigma_{D\,zul}$ which are as a rule smaller than σ_{zul} should be compared with σ_{zul} the following relation applies

$$\gamma \cdot \sigma_{D\,zul} = \sigma_{zul} \tag{17}$$

out of which we receive

$$\gamma \cdot \frac{\max M_I}{W} = \sigma_{zul}. \tag{18}$$

The combination of the formulae (16) and (17) gives

$$\gamma = \frac{\sigma_{zul}}{\sigma_{D\,zul}} = \frac{\sigma_{zul}}{\sigma_{U\,zul}} \cdot \left(1 - \operatorname{tg}\alpha \cdot \frac{\min M_I}{\max M_I}\right) \tag{19}$$

in which α is a linear function of $\frac{\min M_I}{\max M_I}$. The values γ can be calculated with $\sigma_{U\,zul}$ and $\sigma_{W\,zul}$ for any conditions guiding the ratio $\frac{\min M_I}{\max M_I}$. The simplest way is as below for bridges in steel 37 we receive

for $\min M_I = -\max M_I$ . . . . . $\gamma_{-1} = \frac{14}{10{,}8} = \sim 1{,}3$

and for $\min M_I = 0$ . . . $\sigma^{Ia}_{U\,zul} = 14\ \text{kg/mm}^2$, hence $\gamma_o = \frac{14}{14} = 1$.

In general we have

$$\gamma = a + b \cdot \frac{\min M_I}{\max M_I}$$

and for $\min M_I = 0$ we receive $\alpha = 1$

and for $\frac{\min M_I}{\max M_I} = -1$, the relation $1{,}3 = 1 + b\,(-1)$ is obtained

out of which $b = -0{,}3$

therefore for steel 37

$$\gamma = 1 - 0{,}3 \cdot \frac{\min M_I}{\max M_I} \quad \text{(same as for riveted bridges).} \tag{20}$$

With a similar calculation the values γ are obtained for steel 52, which are

$$\text{for heavy traffic} \quad = \gamma = 1{,}235 - 1{,}237 \cdot \frac{\min M_I}{\max M_I} \tag{21}$$

$$\text{for light traffic} \quad = \gamma = 1{,}105 - 1{,}102 \cdot \frac{\min M_I}{\max M_I}. \tag{22}$$

The respective diagrams are shown in fig. 34 and 35.

According to the definition for γ (formulae 17, 18) follows that the value for γ can never be less than 1, even if the calculation should give results to the contrary (zone of surging-loads).

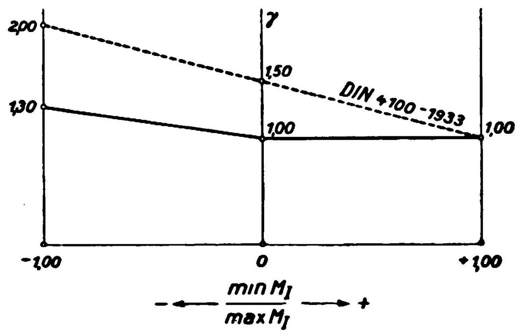


Fig. 34.
γ-lines for St. 37.

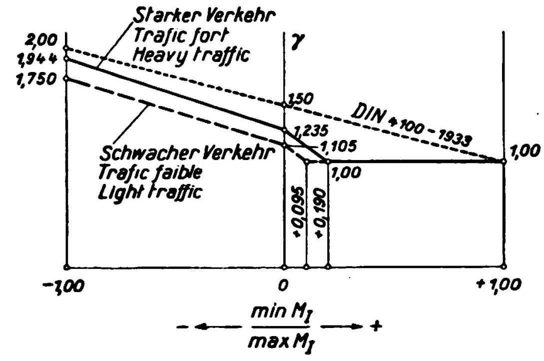


Fig. 35.
γ-lines for St. 52.

3) *Coefficients* α.

For girders in bending the really permissible stress (not the assumed stress) in stipulated by the following term

$$\sigma_{D\ zul} = \frac{\max M_I}{W_n} \qquad (7\,V)$$

(according to stress limitation diagrams). This term put into equation 5 V on page 28 we receive

$$\alpha = \gamma \cdot \frac{\sigma_{D\ zul}}{\sigma_{zul}}. \qquad (8\,V)$$

According to exact calculation the diagrams for the α-values, dependent on the ratio $\frac{\min M_I}{\max M_I}$ would be slightly curved lines which, however, are replaced with sufficient accuracy by straight lines between two limits, for instance between

$$\alpha_{-I} \quad \text{for} \ \frac{\min M_I}{\max M_I} = -1$$

and

$$\alpha_0 \quad \text{for} \ \frac{\min M_I}{\max M_I} = 0.$$

In general α can be expressed by the equation

$$\alpha = a + b \cdot \frac{\min M_I}{\max M_I}.$$

With this expression have been calculated the α-values in table 2 V page 32 for st. 37 (the table for st. 52 is similar). The above will be explained in the following example.

The butt welds are such as can be rewelded at the root.

The highest stresses are in tension (line IIa).

The previous regulations prescribed under all circumstances the provision of additional fish-plates for members in tension, even if the flange plates of girders were butt-welded. The fish plates could be fixed with fillet welds or by rivetage.

In table 2, page 6, is shown that for end-fillet welds as well as for the beginning or ending of side-fillet welds the fatigue strength is considerably lower (only about half) that of butt-welds, and as the chords of girders even at the beginning of cover plates shall not be stressed higher than fillet welds, it results in unsuitable and uneconomical constructions, this particularly for statically indeterminate constructions. Similar conditions also exist for rivetage which does not allow satisfactory results on account of the weakening of the sections due to rivet holes. These points led to the adoption of a solution which proved an enormous progress: Butt-welds also for members in tension without extra cover plates to the joints. It is obvious that in such cases only butt-welds of the best workmanship are allowed, with whenever possible rewelded roots and careful machining of the welds to establish a gradual transition from weld to plates (the welds are allowed to be worked off till they are even with the plates). Hollows in the surface of the weld are not to be permitted. The total weakening of the plates (after grinding) shall not be more than 5 % of the thickness of the plates. From table II, line 3, we learn that the fatigue strength for butt-welds if placed under 45^0 is considerably higher ($\sigma_{U\,zul} = 22$ kg/mm² against 18 kg/mm²), therefore all such butt-welds if in tension should be arranged under 45^0.

4) For the $\sigma_{D\,zul}$-line IIa, (jointed members in tension in the vicinity of butt-welds and butt-welds themselves) for steel 37 we have

$$\gamma_{-I} = 1{,}30 \quad \text{(fig. 34)}$$
$$\sigma^{IIa}_{W\,zul} = 8{,}6\ \text{kg/mm}^2$$

or according to formula 8 V

$$\alpha_{-I} = \frac{\gamma\,\sigma_{D\,zul}}{\sigma_{zul}} = \frac{1{,}30 \cdot 8{,}6}{14} = \sim 0{,}8$$
$$\gamma_0 = 1$$
$$\sigma^{IIa}_{U\,zul} = 11{,}2\ \text{kg/mm}^2 \qquad \text{(fig. 1 V)}$$

hence

$$\alpha_0 = 1 \cdot \frac{11{,}2}{14} = 0{,}8$$

therefore for both ranges we receive

$$\alpha = 0{,}8 \quad \text{(see table 2 V line 4)}$$

(The actual stress will not be more than

$$\sigma_r = \frac{\alpha}{\gamma} \cdot \sigma_{zul} = \frac{0{,}8}{1{,}3} \cdot 1400 = 860\ \text{kg/cm}^2$$

whilst the alternating strength according to page 6, table II, line 3, can be taken at

$$\sigma_W = 1300\ \text{kg/cm}^2)$$

In the zone for compression we have for line IIb the value

$$\gamma_{-1} = 1{,}30$$
$$\sigma^{IIb}_{W\,zul} = -\ 8{,}6\ \mathrm{kg/mm^2} \qquad \text{(fig. 1 V)}$$

hence
$$\alpha_{-1} = \frac{1{,}3 \cdot 8{,}6}{14} = \sim 0{,}8 \ \text{ as for IIa}$$

Further it is
$$\gamma_0 = 1{,}0$$
$$\sigma^{IIb}_{U\,zul} = -\ 14{,}0\ \mathrm{kg/mm^2}$$

therefore
$$\alpha_0 = 1{,}0 \cdot \frac{14}{14} = 1$$

(for the whole zone of surging loads, see table 2 V, line 5).
The value α can generally be expressed by

$$\alpha = a + b \cdot \frac{\min M_I}{\max M_I}$$

and for
$$\min M_I = 0 \text{ we receive}$$
$$\alpha = 1$$

while for
$$\frac{\min M_I}{\max M_I} = -1 \qquad \text{we again receive } \alpha_{-1} = 0{,}8$$

hence $0{,}8 = 1 + 6\ (-1)$ or $b = 0{,}2$
therefore

$$\alpha = 1 + 0{,}2 \cdot \frac{\min M_I}{\max M_I} \qquad \text{(Zone of surging loads)} \tag{23}$$

see table 2 V, line 5)
Remarks on table 2 V, line 18. On account of the through fillet welds of flange plates and cover plates being interrupted at the connection between longitudinal beams and cross girders, it is necessary to reduce the permissible stresses for the flange plates of the longitudinal beams according to line 14 to 17 of table 2 V, on account of the notching action produced at the places where these fillet welds are interrupted.

The machining of butt-welded joints in web plates is necessary at those places where the difference in stress between upper and lower stress is more than 11,2 $\mathrm{kg/mm^2}$ without taking into account the γ-values. The machining of such welds is necessary as untreated welds are not permitted to be stressed higher. The α-values for bridges in steel 52 and light traffic are the same as for heavy traffic, only the γ-values are different for these cases.

F. Structural details and execution.

1) It is necessary right from the beginning of a design to bear in mind that welded joints should be accessible and that the welding outfit properly be held in position for working. Overhead welding operations should be avoided.

It is best to try for welding operations to be carried out in a horizontal position. The stipulation that welding operations should be carried out in horizontal position is particularly important in bridge building since excellent workmanship is only possible if the welder is given all facilities.

Structural steel firms concerned with the welding of bridges should be equipped with the necessary handling devices to permit the parts to be brought in positions for easy welding.

2) Joints should be avoided if economy permits. Latest practice has abandoned the use of flange plates of cut length adapted to the modulus of section of girders, preference is given nowadays to through-plates of one thickness so doing away with a number of joints. Thick plates have also the advantage that they do not warp like thin plates. Plates up to 15 tons weight have been used without noticeably increasing the total costs.

3) Too many welds closed together should be avoided to have.

4) Interrupted welds and slot welds are not allowed in bridge building. Interrupted welds have a strongly reduced fatigue strength on account of the notching influence at the beginning and at the ends of the welds[15]. Slot welds have to be considered the same as interrupted fillet welds.

5) Fillet welds in general should have equal sides and not be thicker than required by calculation if practical reasons of welding do not demand the contrary. End fillet welds can be carried out with unequal sides permitting in this way an easier flow of the forces.

6) At all places where fillet welds start or end a gradual transition should be established whenever possible, i. e. the ends should be machined to permit the application of the higher values given in tables 2 V and 3 V, lines 16 and 17 (lines 6a and 6b of the diagrams for stress limitations fig. 1 V and 2 V).

7) Stiffeners and girder connections subject to compression only are permitted to be welded together with the girder flanges in compression (exceptions against the rules of table 2 V and 3 V only for girder flanges, which apart from compression may also receive tension, but this exception only if under consideration of the values α according to table 2 V and 3 V, lines 14 and 15, the permissible bending stresses in the flanges are not overstepped. Otherwise tight-fitting distance pieces must be provided between stiffeners and the chord in tension. Fig. 12 V). Under all circumstances it is necessary that no gaps exist between the flanges of the girder and the stiffener.

The stiffeners and other connections attached to the flanges of girders must be provided with chamfered corners in such a way as to permit the main weld between web plate and flanges to pass without interruption and to allow for examination.

Plate girders with web plates of $\geqq 1$ m in width and girders with big shear forces require web plates free from warps[16]. Girders where the web

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[15] See *Hochheim:* Mitteilungen aus den Forschungsanstalten der Gute-Hoffnungshütte 1932, page 225, 1. A A girder with interrupted welds and extreme fibre stresses of 1560 kg/cm² stood only 60 000 loading repetitions, whilst a similar girder but with through-welds did not show ang signs of destruction even for 2.10^6 loading repetitions.

[16] See *Schaper:* Grundlagen des Stahlbaues, page 98, Berlin 1933, Wilhelm Ernst & Son, Editors.

Table 2 V. Coefficients α for Steel 37.

1 No.	2 Structural detail and type of weld	3 Kind of stressing	4 Range of alternating efforts	5 α values Range of surging efforts	6 Remarks
1	Through-members and cover plates *	Tension	1,0	1,0	fig. Ia
2		Compression	1,0	1,0	fig. Ib
3		Shear	0,8	0,8	see line 18*
4	Jointed parts with butt-welds. Re-welding of roots: possible	max. stress tension (+)	0,8	0,8	fig. II a
5		max. stress compressive (—)	$\alpha = 1 + 0{,}2 \frac{\min M_I}{\max M_I}$	1,0	fig. II b
6	not possible	max. stress tension (+)	$\alpha = 0{,}57 + 0{,}11 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0 \leqq 0{,}29$ $\alpha = 0{,}57 + 0{,}79 \frac{\min M_I}{\max M_I}$; f. $\frac{\min M_I}{\max M_I} \geqq 0{,}29$ $\alpha = 0{,}8$	fig. III a
7		max. stress compression (—)	$\alpha = 0{,}71 + 0{,}25 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0 \leqq 0{,}11$ $\alpha = 0{,}71 + 0{,}82 \frac{\min M_I}{\max M_I}$; f. $\frac{\min M_I}{\max M_I} \geqq 0{,}11$ $\alpha = 0{,}8$	fig. III b
8	Through butt - or fillet welds connecting web plate and chords	shear $\sigma = \frac{1}{\alpha}\left[\frac{\sigma_I}{2} + \frac{1}{2}\sqrt{\sigma_I^2 + 4\tau_I^2}\right] \leqq \sigma_{zul.}$	$\alpha = 1{,}1 + 0{,}1 \frac{\min M_I}{\max M_I}$	1,1	fig. IV a and IV b
9	welds and web plate at the connection between web plate and chord	shear $\tau_1' = \frac{\gamma \max Q_{I x} S}{\alpha J t} \leqq \sigma_{zul}$	0,65	0,65	

10	Butt-weld of web plate connection		main stress (same formula as in line 8	1,0	1,0		
11			shear stress $\tau_I' = \frac{\gamma \max Q_{Ix}}{a\ t\ h_s} \leqq \sigma_{zul}$	0,65	0,65		
12	fillet-welds for girder connections stiff in bending		Main stress $\sigma = \frac{1}{a} \sqrt{\sigma_I^2 + \tau_I^2} \leqq \sigma_{zul}$	0,75	0,75		
13			shear stress $\tau_I' = \frac{\gamma \max A_I}{a\ \Sigma\ (al)} \leqq \sigma_{zul}$	0,65	0,65		
14 and 15	Structural parts near endfillet-welds and at places where side-fillet welds start or end fillet-welds to be calculated according to line 19	Terminals of end fillets and side fillets un-treated	max. tress tension (+) or compression (—)	$\alpha = 0{,}71 + 0{,}15 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0 \leqq 0{,}29$ $\alpha = 0{,}71 + 1{,}0 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0{,}29$ $\alpha = 1{,}0$	fig. V a, V b
16		as above but careful machining of the ends	max. stress tension (+)	$\alpha = 0{,}93 + 0{,}13 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0 \leqq 0{,}07$ $\alpha = 0{,}93 + 1{,}0 \frac{\min M_I}{\max M_I}$	for $\frac{\min M_I}{\max M_I} \geqq 0{,}07$ $\alpha = 1{,}0$	fig. VI a
17			max. stress compression (—)	$\alpha = 1 + 0{,}2 \frac{\min M_I}{\max M_I}$	1,0		fig. VI b
18	cover plates and through plates for longitudinal decking beams if sidefillet welds interrupted at connections.		same as for lines 14 to 17	same as for lines 14 to 17	same as for lines 14 to 17		
19	fillet welds		any kind of stressing with exception of main stresses (line 8) and tension or compression in the direction of the weld.	0,65	0,65		
20	The butt-welds of web plates should be x-rayed, they require tooling at those places (to establish a gradual transition between weld and plate) where a difference between upper and lower stress values exists: $\sigma_o - \sigma_u \geqq 0{,}8 \cdot 1400 \geqq 1120$ kg/cm²						

$$\sigma_o = \frac{\max M_I}{W_n} \qquad \sigma_u = \frac{\min M_I}{W_n}$$

The corresponding Table for steel 52 is similar.

plates are not specially examined as regards the evenness of the surfaces should be provided with stiffeners not farther apart than 1,30 m.

If the observance of par. 7 of the regulation should lead to inconveniences, connections of high girders may become questionable for riveted girders as well. (Rivetage should not be principally barred from welded bridges, but should be permitted if connected with advantages.)

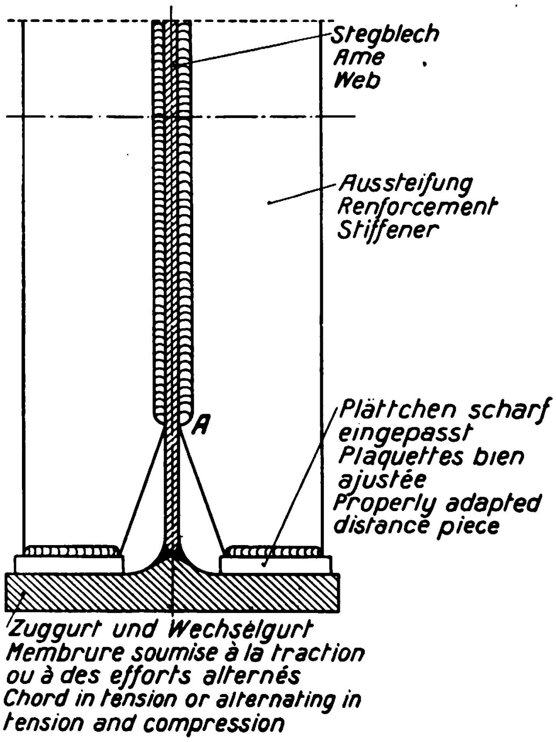

Fig. 12 V.

8) At all places where point loads have to be transmitted, stiffeners should be provided.

9) The minimum thickness for load carrying fillet welds should be a = 3,5 mm (for stiffeners a thickness a = 3,0 mm is permissible). The thickness of fillet welds in general should not be more than $a = 0{,}7\ t_1$, where t_1 represents the thickness of the thinnest plate, flanges or legs of rolled sections, of the welded connection. (Fig. 13 V, 14 V and 15 V.) Deviations from this rule are only permitted if the connection cannot be carried out by other means.

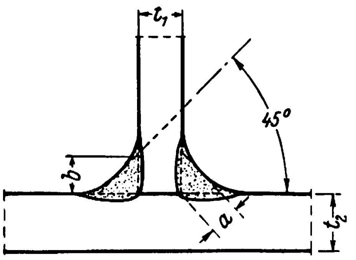

Fig. 13 V.

$t_1 < t_2$, $a \leqq 0{,}7\ t_1$
(exceptionally $a = t_1$).

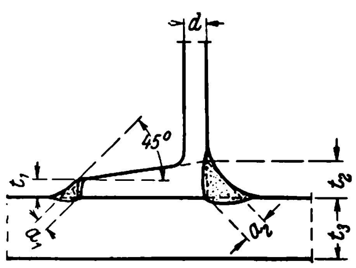

Fig. 14 V.

(Tapered flange).
For $t_1 < t_3$, $a \leqq 0.7\ t_1$, for $t_2 < t_3$ $a_2 \leqq 0.7\ t_2$.

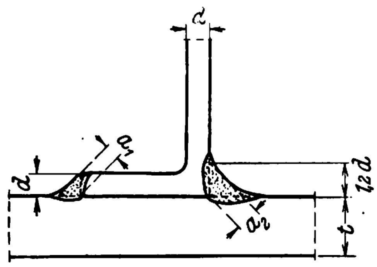

Fig. 15 V.

(Parallel flange).
For $d < t$, $a_1 \leqq 0.7\ d$, for $1.2\ d < t$ $a_2 \leqq 0.7 \cdot 1.2\ d$.

10) Thick flange plates can be jointed employing U-shaped welds, fig. 16a V and 16b V.

Trials with U-jointed plates up to 100 mm in thickness in pulsator machines have given equally good results as for V- or X-jointed plates of less thickness. Example as for fig. 36.

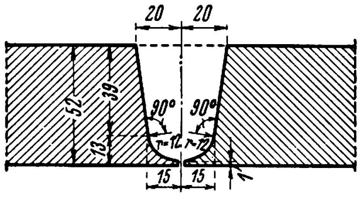

Fig. 36.
Rewelding, example of a U-shaped weld for a plate 52 mm thick.

11) Butt welds of flange plates should have a cross section symmetrical as near as possible to the centre line of gravity of these plates (Fig. 16a V, Fig. 16b V, Fig. 17 V).

U-shaped sections of welds can be regarded as symmetrical sections.

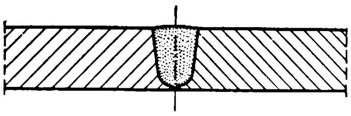
Fig. 16 a V.

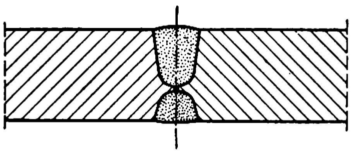
Fig. 16 b V.

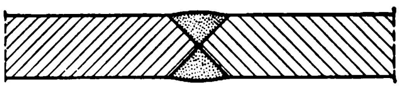
Fig. 17 V.

12) If two flange plates welded and acting together should be jointed, the joint should be for both plates in the same position to enable the roots of both to be rewelded.

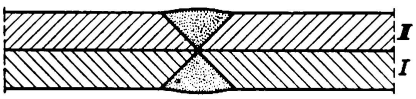

Fig. 18 V.
Two V-shaped welds.

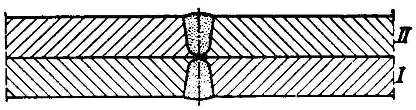

Fig. 19 V.
Two U-shaped welds.

It is important to note that with staggered joints for such plates possibilities exist of causing damage to the lower plate while welding the root of the joint in the upper plate.

13) Chord plates in compression, not in direct and through connection with the web plate, should not be wider than 30 times the thickness of these plates, if a larger width cannot be avoided the flange plate should be riveted or bolted to prevent warping (no deduction is necessary in the compressive chord due to rivet or bolt holes).

14) If the change in thickness of chord plates is necessary, the stepping down from the thicker to the thinner plate should be gradual. Fig. 20 V. This applies also for the transition of thinner web plates to thicker plates.

15) The butt-welded joints of web plates require the roots to be rewelded and require tooling of the transition between weld metal and plate according to the stipulations of table 2 V and 3 V, line 20.

16) Erection holes should be shown in the drawings and should be so arranged that highly stressed sections are not weakened.

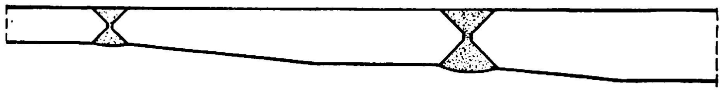

Fig. 20 V.

Flange plates changing in thickness.

Chords in tension should not receive end-fillet welds right angled to the direction of stressing. If unavoidable or where side fillet welds start or end, the welds should be machined to effect a gradual transition between weld and plate, this on account of the α-values (see par. 6).

As lattice girder bridges are not permitted to be welded up to now, welding may still enter into account in case of reinforcing riveted lattice girders. This has the advantage that the girders do not require propping. The calculation in such cases should be such that the rivets carry all dead weight including the dead weight of the reinforcement and that the welded connections are strong enough to carrry all live loads. Should this not be possible then at least $^2/_3$ of the live load must be carried by the welded connections and the balance of the live load to be taken over by rivets[17].

While carrying out such reinforcements to lattice girders the development of stresses due to shrinkage of welds has to be carefully watched.

17) The weld of all joints in chord plates and web plates require to have a gradual transition from weld to plate provided the difference between the highest and the smallest bending stress $\sigma_o - \sigma_u \geqq 11{,}2$ kg/mm². This rule also applies to all other important butt-welds, if specially marked on the working drawings. The transitions at a and b should be gradual and carried out by grinding. Grooves right angled to the direction of stressing should not occur, the surface at such important places must be smooth and free from hollows. If welding causes hollows in the plate or weld metal, the weld has to be removed in workmanlike manner and re-welding and re-machining is required. If by such a procedure the parent metal left and right of a and b requires to be replaced by weld metal this is of no importance, only the gradual transition with a smooth surface is of absolute necessity. Reduction in thickness of the plates up to 5 % is permitted. Grinding at A and B leaving the convex portion of the weld can be replaced by complete removal of the protruding portion of the weld if carried out by grinding in the direction of the acting stresses. The surface must be free from flaws. (Fig. 25 V.)

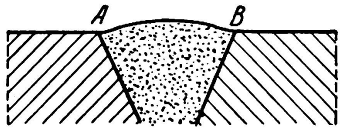

Fig. 25 V.

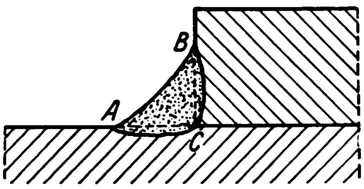

Fig. 26 V.

[17] The acceptability of this regulation has been sufficiently proved by static and fatigue tests carried out at the Governmental Test House in Dahlem. See *Kommerell* and *Bierrett:* Über die statische Festigkeit und die Dauerfestigkeit genieteter, vorbelasteter und unter Vorlast durch Schweißung verstärkter Stabanschlüsse. Stahlbau 1934, page 81.

18) The type and nature of welding has to be shown in the working drawings, for instance: butt weld 1st quality, machining of surface.

19) The machining of the surface of through butt or fillet welds is not necessarily required.

20) All fillet welds should have a fusion area reaching well into the root of the fillet C (Fig. 26 V). An inroad too deep under the surface of the plate should be avoided. For end-fillet welds it is important that the welder produces the exact shape of the weld and observes the measurements prescribed. Notches and grooves at A and B are not to be permitted under any circumstances (otherwise removal of bad places, rewelding and re-machining is necessary).

21) All beginnings and ends of fillet welds require machining to establish a gradual transition between weld and plate provided this operation is necessary under consideration of the α-values given in tables 2 V and 3 V, lines 16 and 17. See fig. 27 V. In the working drawings should be mentioned for instance: machining at the beginning of fillet welds.

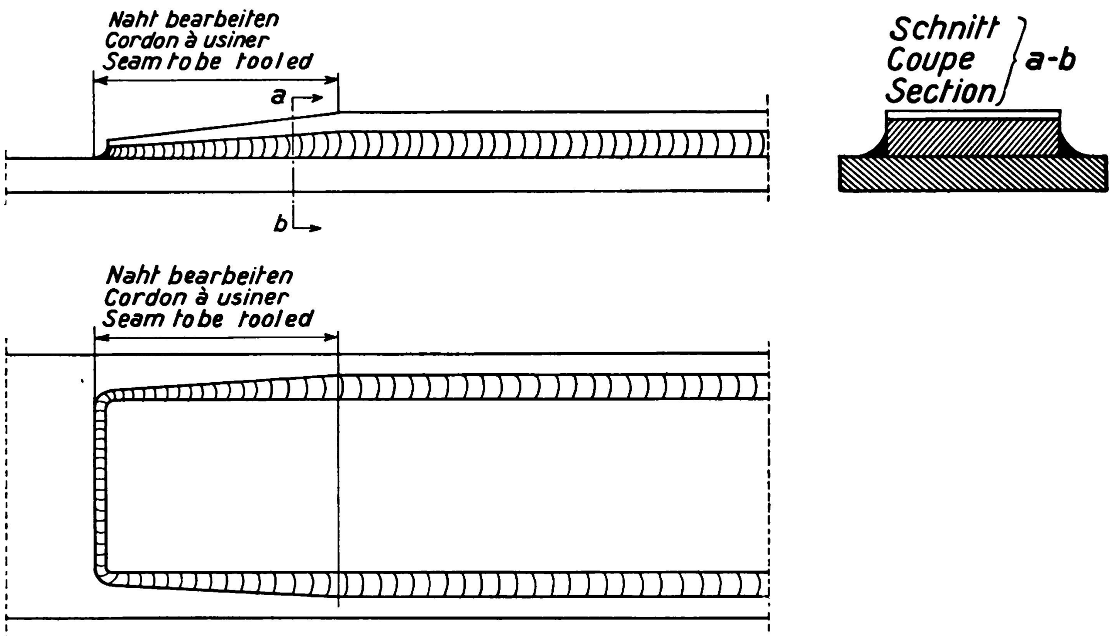

Fig. 27 V.

Establishing of gradual transitions by grinding or machining.

22) Welds not in accordance with the foregoing regulations must carefully be removed with tools not too heavy for the nature of such work and replaced by welds according to regulation. Mended places and their vicinity should be slightly heated with the burner.

23) Simply for the purpose of easy erection no pieces are allowed to be welded to load-carrying parts if not clearly shown in working drawings, even if only temporarily required and to be removed later. If such attachments are necessary little holes should be drilled (whenever possible at places of small stresses), such holes should later on be plugged by rivets or welding.

24) It has to be carefully watched that the structure is not damaged by splashing and drops of weld metal. The burner should only be lit at such places where

welding is required. If damage is done by the possibilities indicated above, causing a notching action in load-carrying parts, such damages have to be made good again by rewelding and carefully polishing the mended places.

ad. 23 and 24. The notching influence due to damages and welded attachments very considerably reduce the fatigue strength. During the execution of a fatigue test carried out in Dahlem, the specimen broke at the place where the welder accidentally touched the plate with the burner.

25) All welding operations should as far as possible be carried out in the workshop. Should the steel work contractor find it more suitable at places to replace welding by rivetage, he has to submit before doing so his proposals to the Directors of the German State Railways for sanction.

26) For the purpose of executing important welds in a horizontal position, suitable handling devices should exist in workshops and at site of erection.

ad. 26. Such handling devices are shown in fig. 37 and 38.

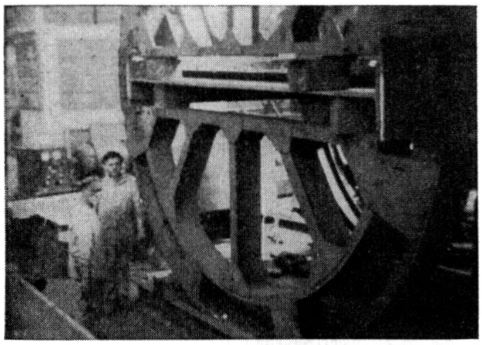

Fig. 37.

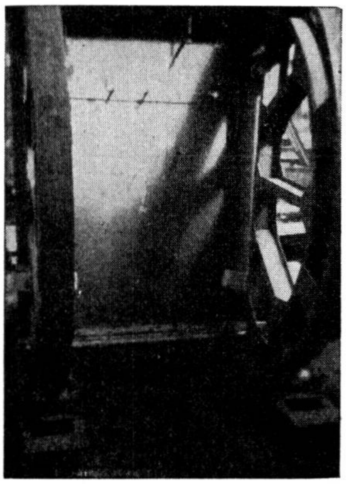

Fig. 38.

27) It is of particular importance that the unavoidable shrinkage of welds should have the possibility of development thus creating the smallest possible stresses due to shrinkage. Pieces to be welded together should therefore not be held too rigidly in position, on the contrary a slight breathing should be allowed to follow the movements of shrinkage.

ad. 27. This rule also applies for erection joints.

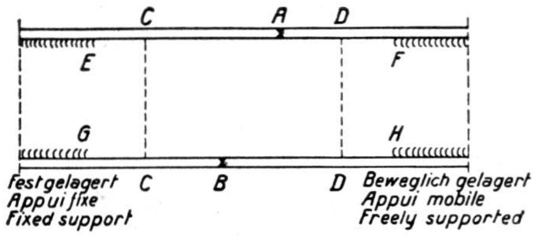

Fig. 39.

If for instance a main girder requires to be welded at A and B it will be so arranged that the web plate is held together at C and D but not welded. Fig. 39. The joints E C, D F, G C and D H will remain unwelded for the time being.

The portion left of the main girder could be rigidly supported and the portion right could be movable, but so that no accidents can occur. The weldings of A and B, which do not cause serious stresses due to shrinkage are done first. After this follows the insertion of the web plate C C D D which will be a little longer than actually required and slightly bulging in such a way that after welding of line C C it comes to fit exactly along line D D.

The butt-welds C C will be done in one operation. No stresses are developed between E C and G C on account of the shrinkage of C C. The welding of line D D causes the slightly bent web plate to straighten. Only after the welding of line E F and G H connecting web plate and chords stresses due to shrinkage are developed in web plates and chords. To distribute the stresses over a sufficient length it was advisable not to weld lines E C, F D etc. before the others. The machining of the various welds takes place only after all welds of the erection joint are complete.

G. Summary.

1) The development in Germany since the Congress of Paris in 1932 of new methods of calculation for welded structures subject to frequently changing loadings is shown. The conclusions drawn by the author from the numerous tests with welded connections explained in the report of the Board of Administrators form the basis of calculation and structural detailing in the "preliminary regulations relating to welded and plated railway bridges of 1935".

2) The dynamic tests carried out with Pulsator machines and swing bridges necessitated the introduction of new terms such as surge-load strength σ_U (Ursprungsfestigkeit), alternating strength σ_W (Wechselfestigkeit), amplitude of stresses σ_w (Schwingweite), which required to be explained and definitions to be given. The curve depicting the fatigue strength can be, with sufficient accuracy, replaced by a straight line, fig. 3, and with the help of angle α, established through a number of test results, the following formula for the alternating strength can be deduced

$$\sigma_W = \frac{\sigma_U}{1 + \operatorname{tg} \alpha} \tag{1}$$

3) In chapter c the chief results of the report of the Board of Administrators are summarized.

a) Welded connections giving good results with statical testing have shown considerably lower values for fatigue tests.

b) Butt-welds on account of the more favourable flow of forces proved superior to fillet welds. Fractures due to fatigue chiefly took place in the parent metal and had their starting point in small hollows at the transition between weld and parent metal (notching action).

d) Surge-load strength for butt-welds where the roots were not rewelded was in value only 70 % of the surge-load strength of butt-welds with rewelded roots.

e) Butt-welds of first quality under 45° to the direction of the acting force produced the surge-load strength $\sigma_U = 22$ kg/mm² compared with $\sigma_U = 18$ kg/mm² for butt-welds of first quality but right angled to the direction of the force.

f) The strength of butt-welds reinforced with additional cover plates did not prove to be higher than that of butt-welds of good quality. It was found that cover plates of dynamically stressed butt-welds may even be the cause of decreased strength.

g) For all fillet welds the fatigue strength decreased if the roots were not properly welded.

h) Light end-fillet welds with a gradual transition from weld metal to plate proved superior to full end-fillet welds.

i) The strength of structural parts is reduced wherever end-fillet welds exist or where structural elements are connected by side-fillet welds to through members; in fact everywhere that side fillet-welds start or end. Higher values of surge-load strength are only obtained if a careful and gradual transition is established between weld metal and parent metal.

k) The cross sectional shape of the weld particularly at the transition of weld metal to plate is of decisive importance for both butt-welds and fillet-welds. The shape of a weld is much more important than the nature of the weld metal.

l) The fatigue test results obtained with swing bridges coincide in general with those obtained by pulsator machines.

m) No important difference in fatigue strength was found for welded connections of steel 37 and steel 52. The qualities of high grade steel entered only into account if subject to pre-stressing.

n) Fatigue tests for through welds stressed longitudinaly gave surge-load strengths of the same values as for butt-welds ($\sigma_u = 16$ to $18\ \mathrm{kg/mm^2}$).

o) Electrodes (welding wires) intended to be used for welding bridges should be subjected to fatigue tests prior to use.

4) The fatigue test values established by numerous tests for the various types of welds and welded connections were as far as necessary complemented with the diagrams for fatigue resistances. Through these fatigue strength values all principle types of connection in the zone of alternating and surging loads can now be regarded as known. These values have been laid down, under consideration of an interval of safety, in the "diagrams of permissible stresses $\sigma_{D\,zul}$ for welded bridges" (diagrams of stress limitations fig. 1 V and 2 V). In these diagrams the upper permissible stress σ_o appears as a function of the lower stresses σ_u.

5) With the intention of not introducing too many values of permissible stresses, which moreover for every type of weld and mode of execution are dependend on the ratio

$$\frac{\sigma_u}{\sigma_o} \quad \text{or} \quad \frac{\min M}{\max M},$$

it was decided, for the purpose of stress calculation, to bring these stresses into conformity with the permissible stresses σ_{zul} ($1400\ \mathrm{kg/cm^2}$ for steel 37 and $2100\ \mathrm{kg/cm^2}$ for steel 52 respectively). For the calculation of cross sectional areas the ideal stress

$$\sigma = \gamma \cdot \frac{\max M}{W} \leq \sigma_{zul} \tag{3}$$

(for instance 1400 kg/cm² for steel 37) has to be proved. The coefficient $\gamma = a + b \cdot \frac{\min M}{\max M} \geqq 1$ represents a dynamical factor by which the maximum bending moment max M must be multiplied to enable the girder to be calculated as if subject to an ordinary static bending moment of the value max M. The values a and b are deduced from the permissible stresses σ_{zul} of the lines 1a and 1b for unjointed through members (diagrams of stress limitations Fig. 1 V and 2 V, page 14). For welded bridges in St. 37 γ is expressed by the formula (Fig. 34)

$$\gamma = 1{,}0 - 0{,}3 \cdot \frac{\min M}{\max M} \tag{20}$$

The reducing coefficients (form factors) α have been introduced with the intention of considering the nature of welds and structural elements in the vicinity of the welded joints. These values for α are derived from the diagrams of stress limitations in such a way that $\alpha = 1$ for unjointed through members and also for members with through-welds (table 2 V, line 1 and 2, page 21). In general for the calculation of welded bridges the following formula has to be used

$$\sigma = \frac{\gamma}{\alpha} \cdot \frac{\max M}{W} \leqq \sigma_{zul} \tag{5 V}$$

(1400 kg/cm² for steel 37).

The values α can be taken from the tables (table 2 V for steel 37) after the ratios $\frac{\min M}{\max M}$ have been found.

6) Chapter F refers to structural details and execution based on the results of tests. The undisturbed flow of forces has a guiding influence for dynamically stressed weld connections. This can be seen especially on well executed butt-welds. Researches have made it possible that nowadays flange plates in tension can be butt-welded without requiring to be reinforced with cover plates. All other knowledge derived from fatigue tests such as machining of welds and joints, for instance the machining of the ends of side-fillet welds or the prohibition of certain stresses in fillet-welds placed right angled to the direction of force in tensile members, has been embodied in the new "Regulations relating to welded and plated railway bridges". These results give the designing engineer the necessary foundation with which to design, calculate and execute welded bridges with safety.

Summary.

Since the Paris Congress of 1932 a new method of calculating welded plated railway bridges has been developed in Germany. The results of fatigue tests as received from Pulsator machines and "swing bridges" have found consideration in the new method of calculation.

Reducing factors, derived from tests were introduced to deal with the nature of welds and the structural parts in the immediate proximity of the welded joints. These values depend, among other factors, on the type of weld (butt weld or fillet weld), and on the position of the weld (e. g. beginning fillet welds, or continuous welds) and for butt welds on whether the root is re-welded or not. The structural detailing and appearance of welded plated railway bridges has been greatly affected by the tests mentioned.

IIIa 2

Fatigue Strength and Safety of Welded Structures (Bridges, Structural Steel Work and Pressure Pipes).

Ermüdungsfestigkeit und Sicherheit geschweißter Konstruktionen (Brücken- und Hochbauten und Druckrohre).

Résistance à la fatigue et sécurité des constructions soudées (Ponts, charpentes, conduites forcées).

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I. Introduction.

Weld metal can be regarded as the product of a steel foundry on very small scale and must therefore be treated fundamentally as though it were cast steel and also judged as such.

Welded joints are far more sensitive to fatigue stress than are building steels assembled by welding or rivet connections. This is due to causes which are unavoidable in practice, such as, slag inclusions, pitting caused by the process of penetration, places in which the welding is not sufficiently binding, very fine cracks due to stressing effects and changes of texture due to heat action, particularly in the zone of transition (penetration zone).

Fatigue tests within varying limits of high stresses offer a very useful means of testing the quality of welded joints both from the metallurgical and from the structural points of view.

When dealing with steels of high carbon content which are more easily influenced by thermal and mechanical factors (e. g. $C = 0.15$ per cent.), the following precautions are justifiable being based on metallurgical knowledge, use and application: use of appropriate special electrodes, preheating of the work pieces, subsequent tempering with the burner, and stress-free annealing of the finished work.

The consideration of all these factors in conjunction with experiences gained, has been of important influence to:

the development of electrodes,
the methods of execution of welded connections,
the design of structural details,
the methods of calculating welded connections and
the testing and supervision of welded structures.

II. Experiments made at the Swiss Federal Institute for the Testing of Materials (E. M. P. A.), 1927—1935.

The results of the most important static and fatigue tests carried out since 1927 by the Federal Institute for the Testing of Materials (E.M.P.A) are published in Report N° 86 of that Institute[1]. The fatigue tests were carried out by means of three pulsating machines of 10, 30 and 60 tons out put respectively and four machines for endurance bending tests made by Messrs. *R. J. Amsler & Co.*, of Schaffhausen, and of an machine for torsion and oscillation-bending made by *C. Schenk*, of Darmstadt.

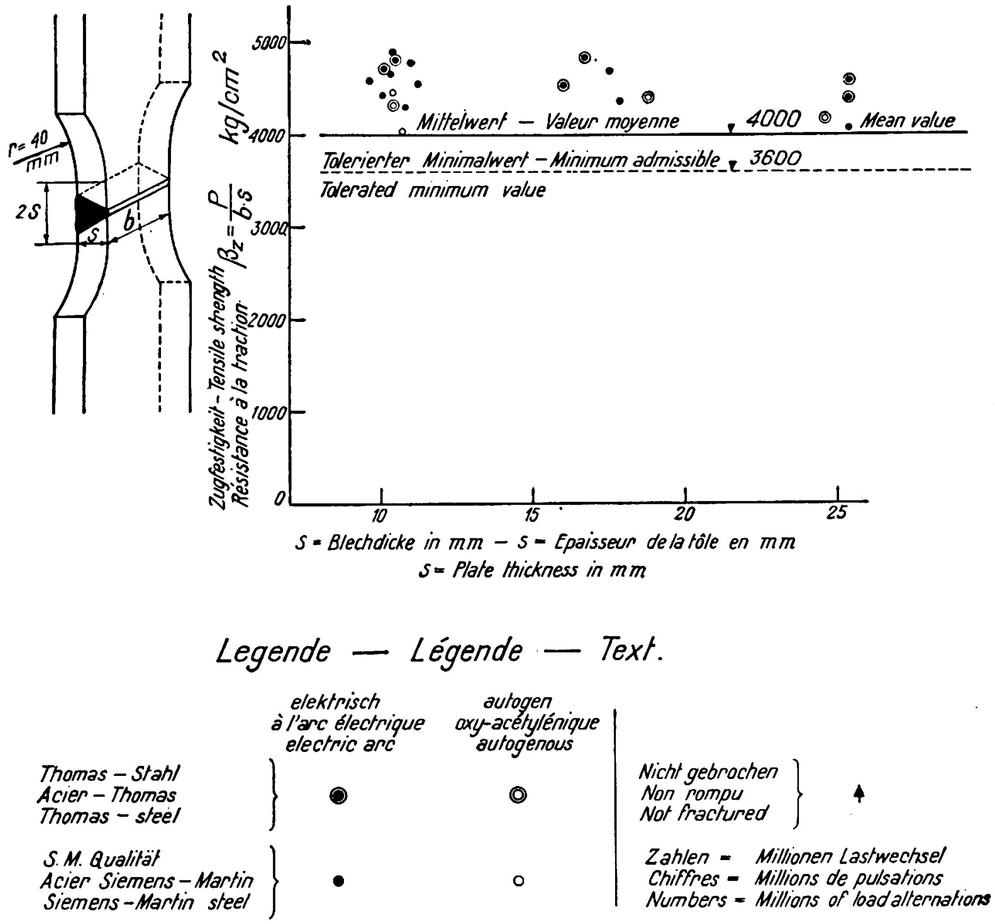

Fig. 1.

Butt weld — Tensile strength.

All the strength values refer to welds of *standard structural steel* with an average tensile strength of $\beta_z \cong 4000\,kg/cm^2$, with a carbon content of $C \cong 0.1\,\%$ and the smallest possible amounts of phosphorous and sulphur ($P + S \leqq 0.1\,\%$) and technically free from slag or eliquations, folds or surface defects (starting hair cracks). The welds were made with *covered electrodes* and direct current (D C); the weld metal was selected to suit to the strength of the steel.

[1] *M. Roš* and *A. Eichinger*, Festigkeit geschweißter Verbindungen (Strength of welded joints). Report N°. 86 of the Swiss Federal Institute for the Testing of Materials, Zurich, March 1935.

The most interesting of these tests are represented in Figs. 1 to 14. The following tables show the mean values of static tensile strengths and of various fatigue strengths. The designations used have the following meaning:

$$\text{Fatigue Strength}\begin{cases}\sigma_D = \text{Oscillation strength}\begin{cases}\sigma\max = +\sigma_D\\ \sigma\min = -\sigma_D\end{cases}\\ \sigma_U = \text{Surge load strength}\begin{cases}\sigma\max = +\sigma_U\\ \sigma\min = 0\end{cases}\\ \frac{1}{2}\sigma_W = \text{Alternating strength}\begin{cases}\sigma\max = +\sigma_W\\ \sigma\min = +\frac{1}{2}\sigma_W\end{cases}\end{cases}$$

visible disturbance of molecular equilibrium . . . . . σ_f = limit of flow
maximum tensile strength, in its relation to the original cross-sectional area . . . . β_z = tensile strength

Table 1.

	Fatigue Strengths Average values in kg/cm²	
	σ_U	$\frac{1}{2}\sigma_W$
Tensile strength		
Butt weld		
Weld (junction of weld and parent metal) (contact area)	1400	2000
Zone of penetration	1600	2300
	— Figs. 2 and 13 —	
Fillet weld		
Weld (junction of weld and parent metal) (contact area)	600	800
Zone of penetration	1000	1500
	— Figs. 4 and 14 —	
Side fillet		
Weld (junction of weld and parent metal)	800	1200
Zone of penetration	1000	1500
	— Figs. 6 and 14 —	
End fillet		
Weld (junction of weld and parent metal)	800	1200
Zone of penetration	1000	1500
	— Figs. 8 and 14 —	

As fillet, side fillet and end fillet welds are at present classified in the same category of practical equality, the following total average figures are stated:

Weld (junction of weld and parent metal)	~ 700	1100
Zone of penetration	~ 1000	1500

The stresses for fillet, side fillet and end fillet welds have always been brought into relation with the sectional area of contact, because fatigue fractures mostly start at this point. As the fatigue fracture may occur along the contact area of the weld or in the zone of penetration, both these places must be examined and the result taken into account.

As the fatigue strength of fillet welds (group) depends on the dimensions of the plates and welds, and also on the resultant values of deformation, the fatigue strengths given in Table 1 and the strength conditions in Table 7, can be considered only as provisionally binding directions. More thorough investigation will be necessary in order to clarify further the influence of length of joint, depth of joint, width of cover plates and thickness of the metal.

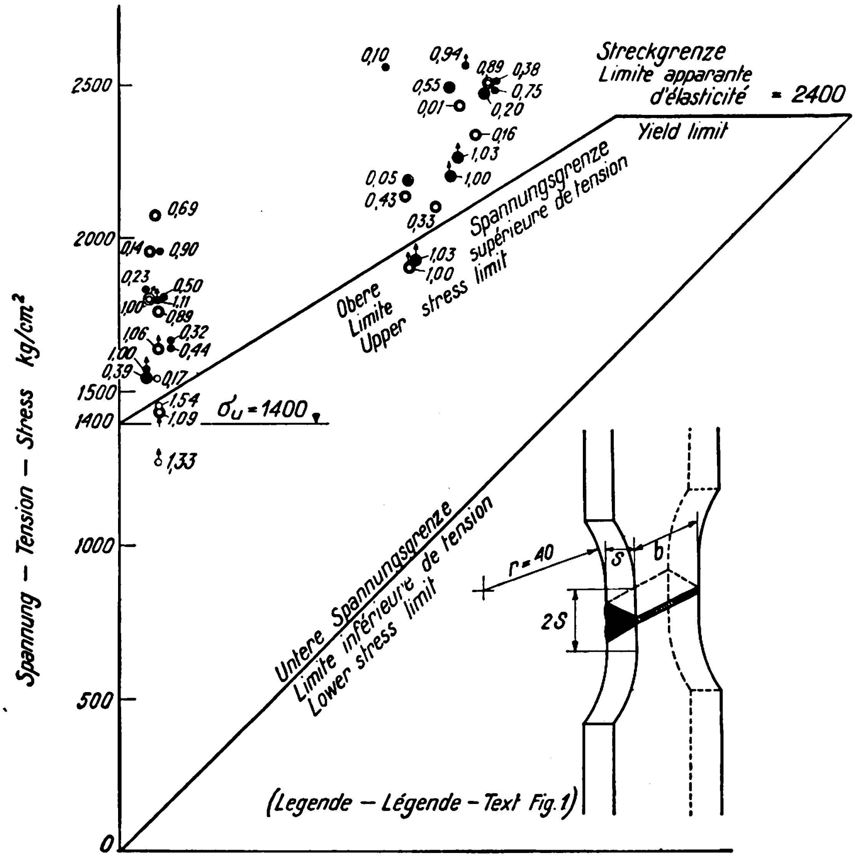

Fig. 2.

Butt weld — Tension-fatigue test.

The combined average values of fatigue strength of butt-welds and of the fillet-weld group for contact areas and the zones of penetration are given in Figs. 13 and 14 in the form of diagrams[2].

[2] The coefficients of resistance to oscillation for tension and compression (volume) could not be determined direct by tests. The fatigue strengths obtained with the "*Schenk*" fatigue machine determined the fatigue strengths for bending and torsion (extreme fibres) resulted in the following ratios:

$$\text{Bending } \frac{\sigma_D}{\sigma_U} = 0{,}7 \qquad \text{— Figs. 9 and 10 —}$$

$$\text{Torsion } \frac{\tau_D}{\tau_U} = 0{,}6 \qquad \text{— Fig. 12 —}$$

The average of these is thus seen to be 0.65. This average of 0.65 was used as the basis for determining the tension-compression-oscillation strength; and for butt-weld therefore, the oscillation strength is: $\sigma_D = 0.65 \cdot 1400 \cong 900\,\text{kg/cm}^2$. The σ_D value calculated in this way agrees very well with the value obtained by *Haigh* for the butt-weld, when using his tension-compression-oscillation machine, this figure being:

$$\frac{\sigma_D}{\sigma_U} = \frac{930}{1470} \cong 0{,}64$$

The average figures of the *static tensile strengths*[3] of butt- and fillet-welds executed in a workmanlike manner, and of the *bending coefficients*[3] are given in the following table[2]. The results of tensile strength tests are shown in Figs. 1, 3, 5 and 7.

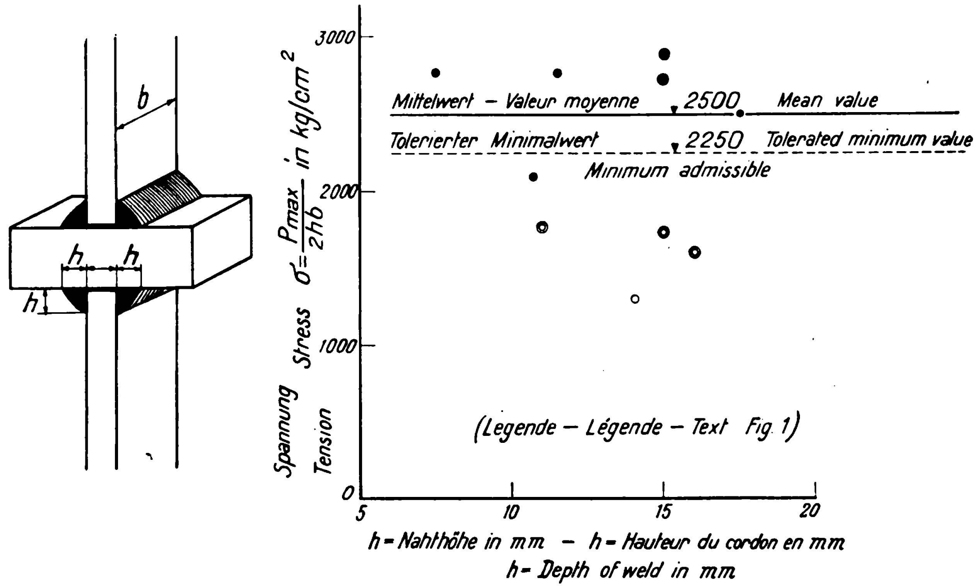

Fig. 3.

Fillet weld — Tensile strength.

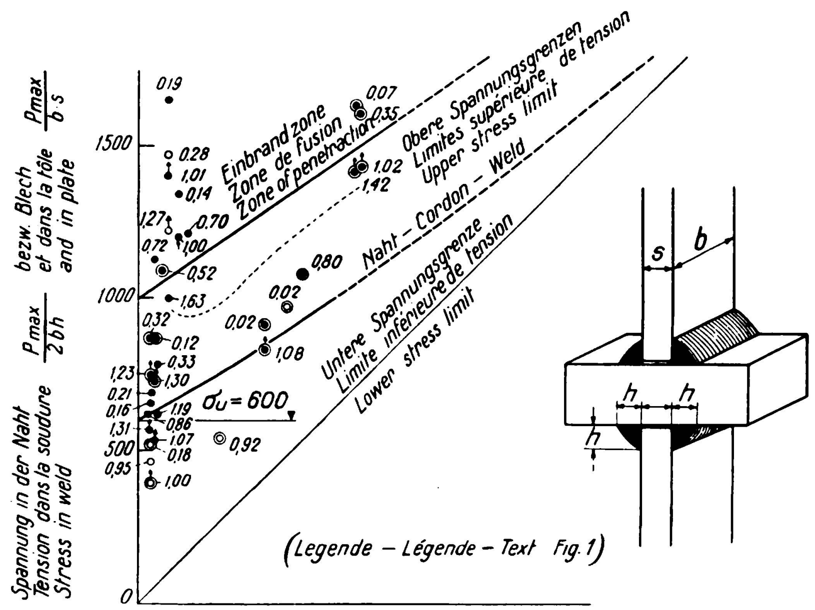

Fig. 4.

Fillet weld — Tension-fatigue test.

[3] The static strengths in Table 2, and the permissible stresses in Table 3 show values which are based on the experiments carried out by the Swiss Federal Institute for the Testing of Materials in the years 1927 to 1935 and have been accepted without any modification in the "New *Regulations for the calculation, execution and upkeep of steel structures placed under the control of the Swiss Confederation.*" These Regulations are dated May 14th 1935.

Table 2.

Tensile Strength	Static strength in kg/cm² Average value	Minimum value
Butt-weld	4000	3600
Fillet weld	2500	2250
Side fillet weld	2500	2250
End fillet weld	3500	3150

Thickness of weld	Bending coefficient: $K = 50 \cdot \frac{s}{r}$ Root of weld in the Pressure zone	Tension zone
< 12 mm	28	20
12—20 mm	20	16
> 20 mm	16	12

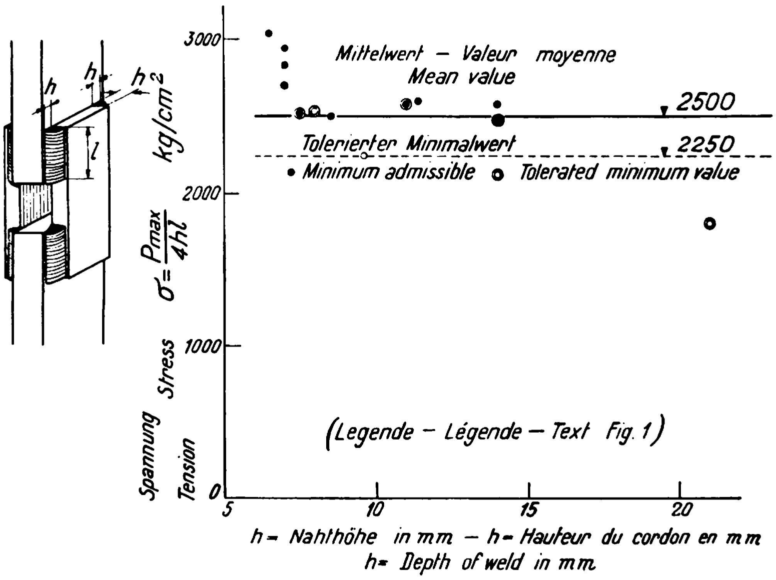

Fig. 5.

Side fillet weld — Tensile Strength.

With increasing carbon content, in order to raise the strength, the precautions, or in other words the difficulties in welding and cutting with the burner are increased. These precautionary measures include:

pre-heating, heating during welding, welding with heavy electrodes and with thicker layers (the latter cannot be complied with for overhead welding):

after completion of welding: annealing at a temperature which is above the upper transformation temperature, (normalisation; in the case of cast steel this is linked up with complete annealing), stress free annealing (up to the lower sub-transformation temperature) and if necessary subsequent tempering with the burner.

The observance of these measures is possible when dealing with pressure piping and cast steel bodies, however it is rarely possible in bridges, and other structural steel work (on account of warping, twisting and the cost incurred).

III. Permissible stresses.

For welded connections of standard structural steel ($\beta_z \cong 4000$ kg/cm²) the following stresses are permissible (kg/cm²):

Table 3.

	Bridges		Structural Engineering [4]	
Butt-welds.				
Tension	$850\left(1+0{,}4\ \frac{A}{B}\right)$	— Fig. 16 —	$1000\left(1+0{,}4\ \frac{A}{B}\right)$	— Fig. 16 —
Compression	$1200\left(1+0{,}3\ \frac{A}{B}\right)$	— Fig. 15 —	$1400\left(1+0{,}3\ \frac{A}{B}\right)$	— Fig. 15 —
Shear	$660\left(1+0{,}4\ \frac{A}{B}\right)$		$770\left(1+0{,}4\ \frac{A}{B}\right)$	
Fillet welds.				
Tension	$425\left(1+0{,}4\ \frac{A}{B}\right)$	— Fig. 17 —	$500\left(1+0{,}4\ \frac{A}{B}\right)$	— Fig. 17 —
Compression	$600\left(1+0{,}3\ \frac{A}{B}\right)$		$700\left(1+0{,}3\ \frac{A}{B}\right)$	
Shear	$480\left(1+0{,}4\ \frac{A}{B}\right)$		$560\left(1+0{,}4\ \frac{A}{B}\right)$	

Here A = the lower and absolutely minimum limit of external force (moment-, axial or shear force),
and B = the upper and absolutely maximum limit of the same forces; the sign (+) changes into (—) when the extreme values are of opposite sense.

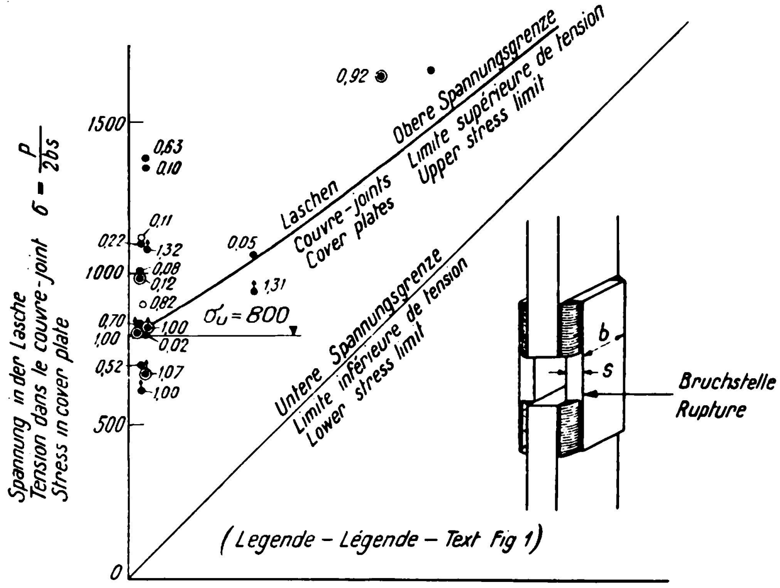

Fig. 6.

Side fillet weld — Tension-fatigue test.

[4] Applicable also to bridges if all the secondary influences resulting from braking, friction, changes of temperature, etc. are taken into consideration. At the same time the permissible stresses given under "Bridges" must not be exceeded for main influences such as dead weight, working and traffic loads, centrifugal forces and dynamic effects.

As compared to riveting, the ratios of permissible stress in welding surge load to rivetage for A = 0 (surge load strength) are.

Table 4.

Ratios[5]

Stressed for:	Riveting	Welding Butt-welds	Welding Fillet-welds
Tension	1.00	0.70	0.35
Compression . . .	1.00	1.00	0.50
Shear	0.58	0.55	—
Pure shear. . . .	0.80	—	0.40

The ratios for the remaining values of $\frac{A}{B}$ are practically of the same magnitude.

[5] The ratios given here correspond to the respective ratios for fatigue strength of welding to riveting. For the selection of ratios of fatigue strengths the fact was of importance that fatigue strength tests allow in a much clearer way with much greater differences to distinguish between properly and incorrectly welded connection, than would be possible by results received from static tests. It has however to be considered that welded structures under practical conditions are subjected to more or less important variations of stressing of different degrees within different limits, on account of alternating stress effects, jerks, interruptions of work, fluctuations in pressure, changes in temperature etc. Welded joints showing small cracks or material containing flaws, behave when stressed mainly by permanent static loads much the same as material which is practically flawless does under repeated alternating stress; the crack gradually spreads until finally fracture takes place.

The following values for surge load strength of welds in relation to rivetage are based on tests carried out by the E. M. P. A.:

	Tension	Compression	Shear
Butt-weld			
unannealed	$\alpha_1 = \frac{1400}{1900} \cong 0{,}7$	$\alpha_1 = 1{,}0$	$\alpha = \frac{1100}{1900} \cong 0{,}55$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	
annealed	$\alpha_1 = \frac{1500}{1900} \cong 0{,}8$		
	$\alpha_2 = \frac{1800}{1900} \cong 0{,}95$		— Fig. 13 —
Fillet, side and end fillet welds.			
	$\alpha_1 = \frac{700}{1900} \cong 0{,}35$	$\alpha_1 = 0{,}5$	$\alpha \cong \frac{750}{1900} = 0{,}4$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	— Fig. 14 —
Zone of penetration.			
Butt-welds	$\alpha_1 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_1 = 1{,}0$	$\alpha \cong 0{,}58$
	$\alpha_2 = \frac{1610}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	
Fillet welds (group)	$\alpha_1 = \frac{1100}{1900} \cong 0{,}6$	$\alpha_1 = 0{,}9$	$\alpha \cong 0{,}53$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	— Fig, 14 and 15 —

For pure *compression* in butt-welds, without danger of buckling, the fatigue strength is considerably higher than for pure tension: the yield limit due to the thermal influence is only slightly lower ($\sigma_f = 2400$ kg/cm² as compared to

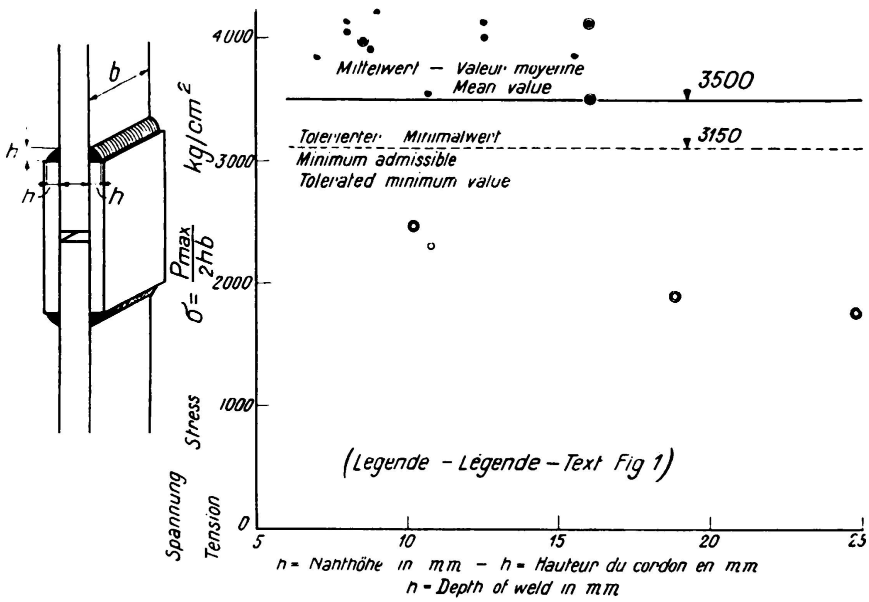

Fig. 7.

Front fillet weld — Tensile strength.

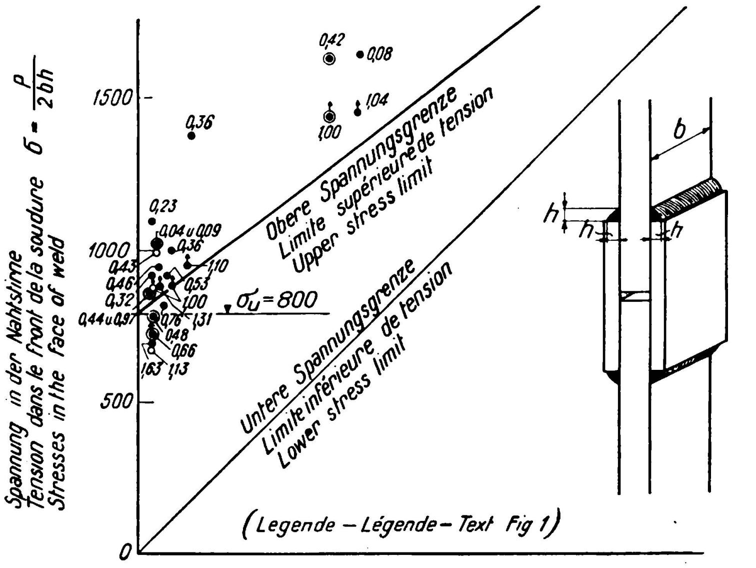

Fig. 8

Front fillet weld — Tension fatigue test.

$\sigma_f = 2600$ kg/cm²) so that it is quite in order to admit a permissible compression stress for riveted connections of standard structural steel of $\beta_z = 4000$ kg/cm².

In the case of shear stresses butt-welds could be tested only in the "*Schenk*" fatigue machine for torsinal fatigue. The torsional fatigue values thus obtained by calculation are too high on account of a more favourable stress distribution over the stressed sectional area. The same position is found with bars stressed in bending. If the fatigue strength ratios for torsion and bending are also applied to butt-welds stressed for shear and tension, the following figures will be obtained:

Table 5.

	Stressing for shear τ_s Fig. 12	Stressing for bending σ_B Figs. 9 and 10	Ratios $\frac{\tau_s}{\sigma_B}$
Oscillation strength σ_D . . .	11	15	0.74
Surge load strength σ_U . . .	18	21	0.86
Alternating strength $^1/_2\, \sigma_W$ . .	29	40	0.73
		Mean value	$\sim$ 0.78

If the ratio between the permissible shear stress and the permissible tensile stress be assumed as equal, the result, according to Table 4, will be:

$$\frac{\tau_{perm}}{\sigma_{perm}} = \frac{0{,}55}{0{,}70} \cong 0{,}78$$

which is in concordance.

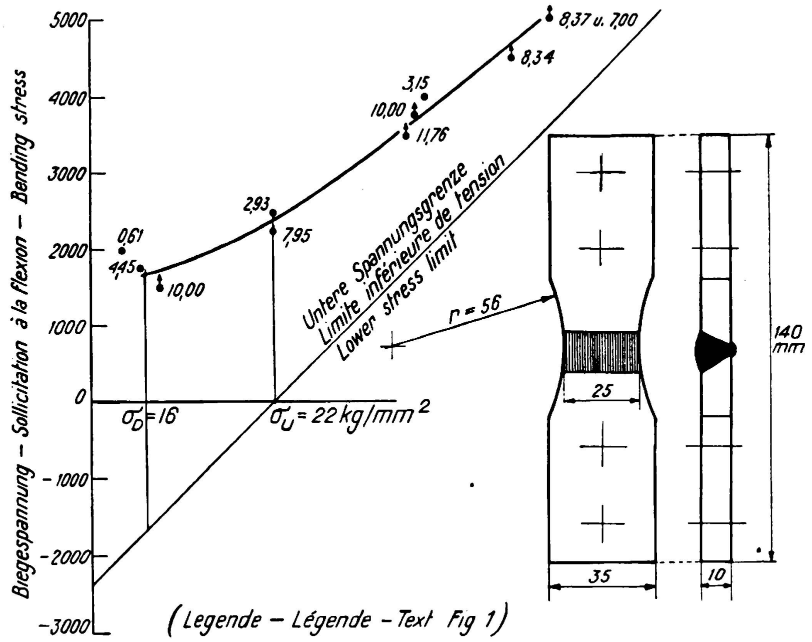

Fig. 9.

Butt weld with bulging surface — Fatigue bending test.

The ratio of permissible stresses of *fillet welds* for tension and compression was taken as equivalent to half (0.5) the permissible tensile stress in butt welds, in correspondence to the ratio for fatigue strength which is practically the same.

Table 6.

	Fatigue strengths kg/cm² σ_U	$\frac{1}{2}\sigma_W$
Butt-welds (Fig. 13)	1400	2000
fillet welds (group) (Fig. 14) .	700	1100
Ratio	0.5	0.55

Fillet welds stressed for shear (e. g. side fillet welds) show higher fatigue strengths for the contact zones, than fillet welds stressed for tension. This fact was taken into account by allowing a shearing stress which was

$$\left(\frac{0{,}40 - 0{,}35}{0{,}35}\right) = \frac{1}{7}$$

higher than that allowed for tension.

When dealing with fillet welds (Fig. 4) note should be taken that steel is stressed in a very unfavourable manner whenever slag bead-lines or folds, are present which might lead to tearing open of the steel. This is a reason for always prescribing the use of flawless material, and the material might be tested by macroscopic texture examination or by x-raying.

Higher values were taken as permissible stresses for fatigue strength in *the zones of penetration* than for the welds (Figs. 13 and 14), also higher than for the welds, so that the coefficients for the permissible stresses in the penetration zone, as compared with riveted joints (Table 7) are higher[6].

Table 7.

Ratios[5] A = 0

Stressed for:	Riveting	Zone of Penetration Butt-weld	Zone of Penetration Fillet welds
Tension . . .	1.0	0.85	0.60
Compression .	1.0	1.0	0.90
Shear	0.58	0.58	—
Pure shear . .	0.80	—	0.53

IV. Safety factor.

On the basis of the figures given in Section II (Experiments made by E.M.P.A.) and Section III (Permissible stresses), the following *calculated factors of safety n_r* are given.

6 High stressing of the fillet welds may have a very unfavourable influence on the fatigue strength of the zone of penetration. This is comprehensible if it be remembered how complicated stress conditions are in that part. The thinner the fillet weld, the higher will be that the accumulation of stresses in the transition zone between parent metal and fillet weld, because the lateral shear stress τ, increases, and this influences the stressing effort since τ' has an important influence to the amount of stressing as will be seen from the formula for comparison stress:

$$\sigma_g = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2 + 3(\tau^2 + \tau'^2)}$$

which is based on the constance of deformation energy

Table 8.

Bridge Construction.

Butt-welds in tension

(Figs. 16 and 17)

Stressed for:	Stresses in kg/cm² σ permissible	σ-fatigue	Factor of safety n_r
Oscillation strength σ_D .	500	900	1.80
Surge load strength σ_U .	850	1400	1.65
Alternating strength $^1/_2\,\sigma_W$	1020	2000	1.95
$\frac{A}{B} = 1$, yield σ_f . . .	1200	2400	2
		Mean value	1.85
			$\sim$ 1.8

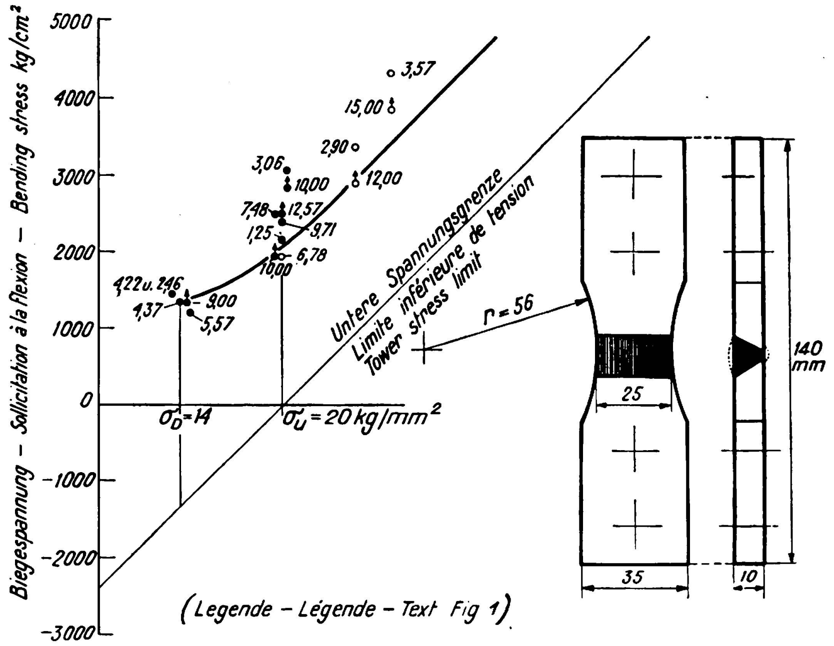

Fig. 10.

Butt weld with protrusion machined off — Fatigue bending test.

Structural Engineering.

Butt-welds in tension

(Figs. 16 and 17)

Stressed for:	Stresses in kg/cm² σ permissible	σ-fatigue	Factor of safety n_r
Oscillation strength σ_D .	600	900	1.50
Surge load strength σ_U .	1000	1400	1.40
Alternating strength $^1/_2\,\sigma_W$	1200	2000	1.65
$\frac{A}{B} = 1$, yield σ_f . . .	1400	2400	1.72
		Mean value	1.57
			$\sim$ 1.5

The calculated safety factors are of practically the same order of value for the contact zones (zones of penetration).

For the compression purposes are given the corresponding calculated safety factors n_r for riveting.

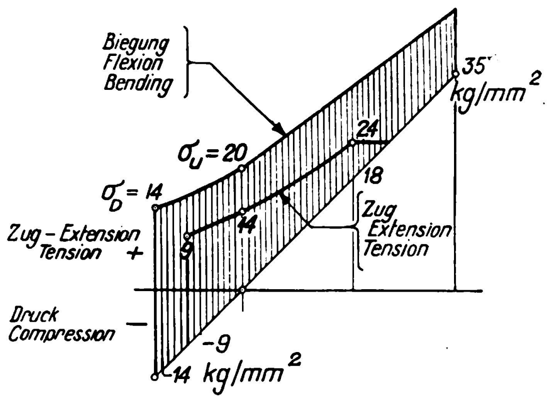

Fig. 11.
Butt welds
Fatigue strength

Table 9.

Bridge Construction.

(Fig. 15)

Stressed for:	Stresses in kg/cm² σ permissible	σ-fatigue	Factor of safety n_r
Oscillation strength σ_D .	840	1300	1.55
Surge load strength σ_U .	1200	1900	1.58
Alternating strength $^1/_2\,\sigma_W$	1380	2400	1.75
$\frac{A}{B} = 1$, yield σ_f . . .	1560	2600	1.67
		Mean value	1.64
			∼ 1.6

Structural Engineering[1].

(Fig. 15)

Oscillation strength σ_D .	980	1300	1.33
Surge load strength σ_U .	1400	1900	1.36
Alternating strength $^1/_2\,\sigma_W$	1610	2400	1.50
$\frac{A}{B} = 1$, yield σ_f . . .	1820	2600	1.43
		Mean value	1.41
			∼ 1.4

From the comparison made between the calculated safety factors for riveting and welding, it will be seen that these are only sligthly higher for welding, on an average about 10 % and can therefore be taken as identical.

The difference between the *real* safety factor and the *calculated* one depends on: the degree of concordance of the following assumption:

the static or dynamic calculation and the actual conditions (external forces, system, stresses),

the design of details,
the technical properties of the material used,
the strictness of control exercised during construction and the workmanship.

Thus *real safety* has to be judged in each individual case according to the circumstances.

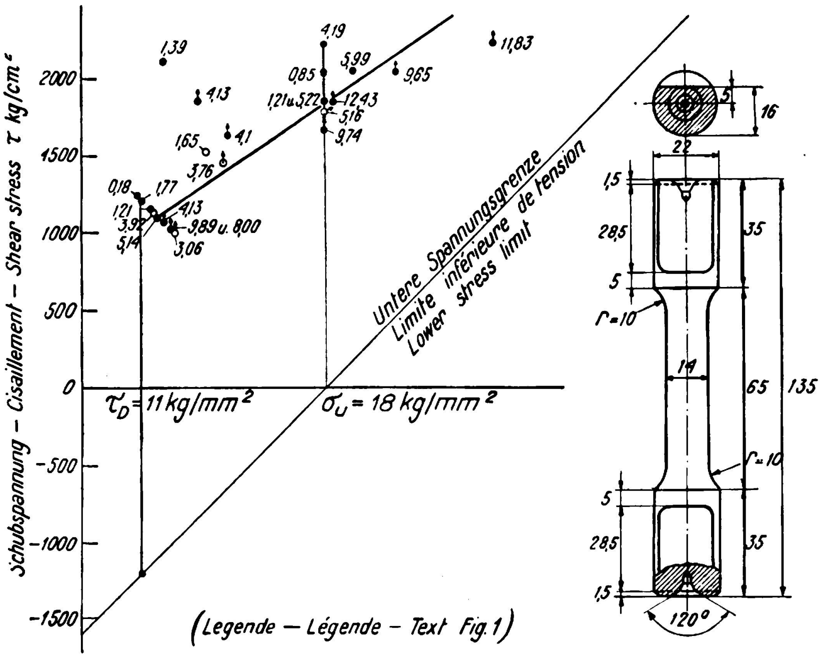

Fig. 12.

Torsion — Fatigue Tests.

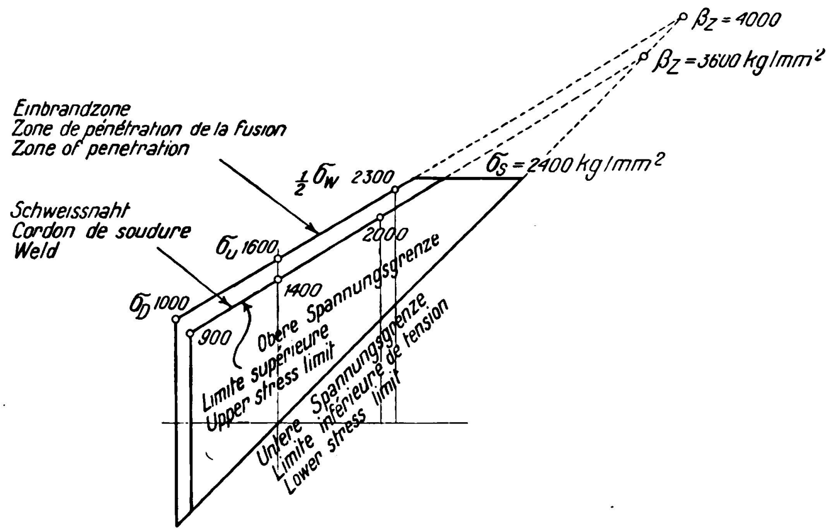

Fig. 13.

Butt welds — Limits of fatigue.

V. Methods of calculation.

When calculating multi-axially stressed members, for instance, in the case of a helical joint on pressure pipes or for a boiler — Fig. 18 — a method of calculation is recommended which admits of determining accurately a stress induced by means of individual component stresses (σ_1, σ_2 and τ).

The theory of constant deformation energy is applicable for stresses up to the rupture strength for practically homogeneous mild steel which is of quasi-isotropic nature as regards its strengths and deformation properties, and which possesses as proved by tests the same strength in both main directions[7].

The comparison stress which refers to the effort in a bi-axially stressed member (σ_1, σ_2 and τ — Fig. 18) is expressed by[8]:

$$\sigma_g = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \cdot \sigma_2 + 3\,\tau^2} \tag{1}$$

The stresses in a weld at right angles and in the longitudinal direction of the weld are not equal. For this reason, and with the aim of maintaining uniform principles, the term expressing the comparison stress of quasi-isotropic bodies

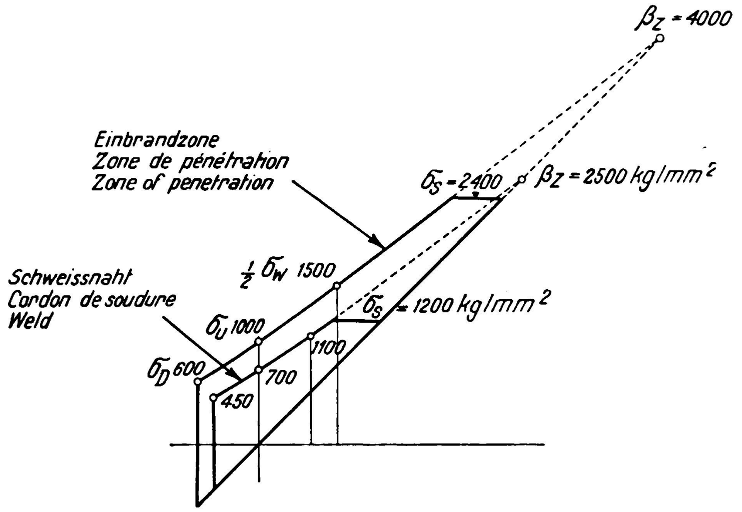

Fig. 14.

Fillet, side fillet and butt welds — Limits of fatigue.

must be remodelled in such a way as to satisfy the an-isotropical property of the welds. This can be done most simply and most accurately by introducing the ratio α expressing the strengths:

$$\alpha = \frac{\text{strength of the welded joint}}{\text{strength of the steel (rivet)}}$$

and at the same time the figures β and γ, determined by tests must be considered which are used in connections with terms σ_1, σ_2 and τ.

[7] *M. Roš and A. Eichinger,* Versuche zur Klärung der Frage der Bruchgefahr, III Metalle. "Tests executed to elucidate the danger of rupture, III Metals" — Report N°. 34 of the E.M.P.A., Zurich, February 1929.

[8]) The theory of the constant deformation energy set up by *Huber-Mises-Hencky* and which has been proved by tests carried out by the E.M.P.A. as constituting an extension of *Mohr's* theory, forms the basis of calculation of uni- or multi-axially stressed members, and is embodied in the New Swiss Federal Regulations relating to the calculation, construction and upkeep of steel structures under the control of the Confederation. The Regulations are dated May 14th- 1935.

Butt-welds.

Case 1.

The normal stresses σ_1 and σ_2 which act at right angles to each other are either both tension, or both compression stresses, then the following relations hold good:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \gamma \cdot \tau^2} \qquad (2)$$

$$\sigma_g = \sqrt{\left(\frac{\sigma_2}{\alpha_2}\right)^2 + \gamma \cdot \tau^2} \qquad \text{— Fig. 19 —} \qquad (3)$$

of which the higher value is decisive.

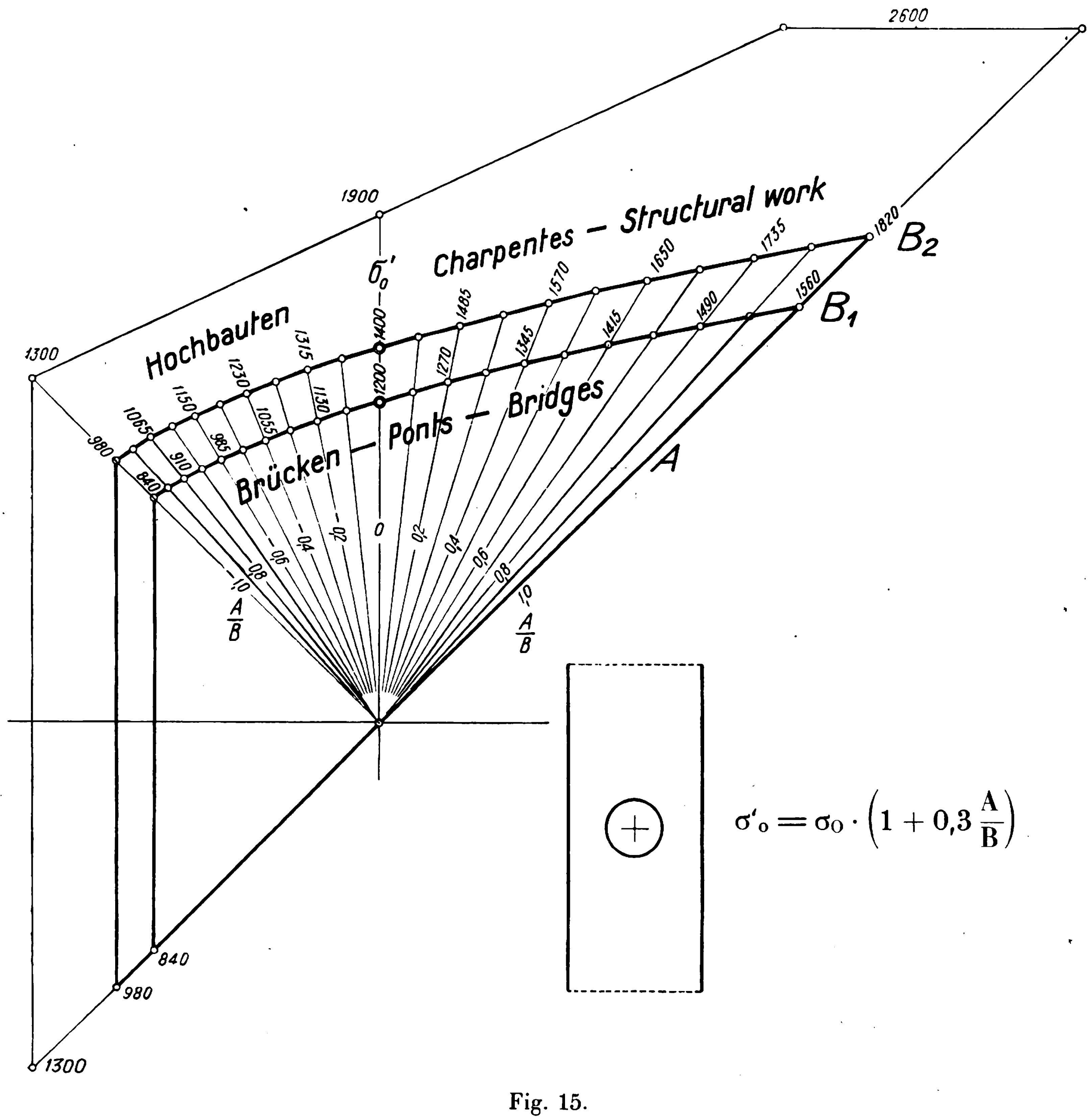

Fig. 15.

Rivetage — Permissible stresses. A = minimum, B = maximum.

Case 2.

The normal stresses σ_1 and σ_2 do not have the same, but opposite signs, and the equation is then[9]:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \left(\frac{\sigma_2}{\alpha_2}\right)^2 + \beta \cdot \left(\frac{\sigma_1 \sigma_2}{\alpha_1 \alpha_2}\right) + \gamma \cdot \tau^2} \qquad (4)$$

(Fig. 19)

The figures for stressing due to surge load strength are

	Tension	Compression	
$\alpha_1 =$	0.75	1.0	(See Table 4)
$\alpha_2 =$	0.85	1.0	
$\beta =$	$^1/_2$		
$\gamma =$	3		

$\alpha_2 = 0.85$ for tension follows from the tests carried out by the E.M.P.A. The average surge load strength for butt-welds stressed longitudinally is 1600 kg/cm², and for riveting this surge load strength was established as 1900 kg/cm² hence:

$$\alpha_2 = \frac{1600}{1900} \cong 0.85.$$

The comparison stresses thus obtained might at most be equivalent to the permissible surge load strength of riveted connections, namely:

$\sigma_{o\,zul} = 1200$ kg/cm² for bridges, or

$\sigma_{o\,zul} = 1400$ kg/cm² for other structural steel work[10].

(zul = permissible.)

See Table 9.

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[9] When comparing the respective figures given in Tables 4 and 7 for shear or punching shear with the corresponding values in the diagrams 19, 20 and 21, then the average value of the diametrically marked figures (under 45°) must be taken.

In order to establish agreement of the figures noted in Figs. 19, 20 and 21 in the lower rigth-hand quadrant under 45° by inserting on the one occasion $\sigma_1 = \sigma_2$ and $\tau = 0$, and the other time by inserting $\sigma_1 = \sigma_2 = 0$, the following eight coefficients have to be used for the particular lower right hand branch of the curve:

In Fig. 19 = 4 instead of 3, therefore 4 τ^2 instead of 3 τ^2
In Fig. 20 = 10 instead of 6, therefore 10 τ^2 instead of 6 τ^2
In Fig. 21 = 4.5 instead of 3.0, therefore 4.5 τ^2 instead of 3.0 τ^2

The values marked were determined by putting $\sigma_1 = \sigma_2$ and $\tau = 0$.

[10] For pressure pipes with butt-welded joints and for straining efforts due to working pressure with an increase of 10% for impact the following permissible stresses apply:

	Quality of steel	
	M I	M II
	$\beta_z = 3500$—4400 kg/cm²	$\beta_z = 4100$—4700 kg/cm²
Pressure pipes:		
longitudinal welds	900	1050
helical welds	1050	1230
Distribution pipes:		
longitudinal welds	800	930
helical welds	930	1080

If these considerations are applied to fillet welds and the zone of penetration, taking into account, however, the respective α ratios (see Tables 4 and 7), the following relations are obtained:

Fillet welds.

Case 3.

The normal stresses σ_1 and σ_2 are either both tensile or both compressive for which the comparison stress is expressed by:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \gamma \cdot \tau^2} \leqq \sigma_{o\,zul} \tag{5}$$

and

$$\sigma_g = \sqrt{\left(\frac{\sigma_2}{\alpha_2}\right)^2 + \gamma \cdot \tau^2} \leqq \sigma_{o\,zul} \tag{6}$$

(Fig. 20)

the higher of the two values for σ_g is decisive.

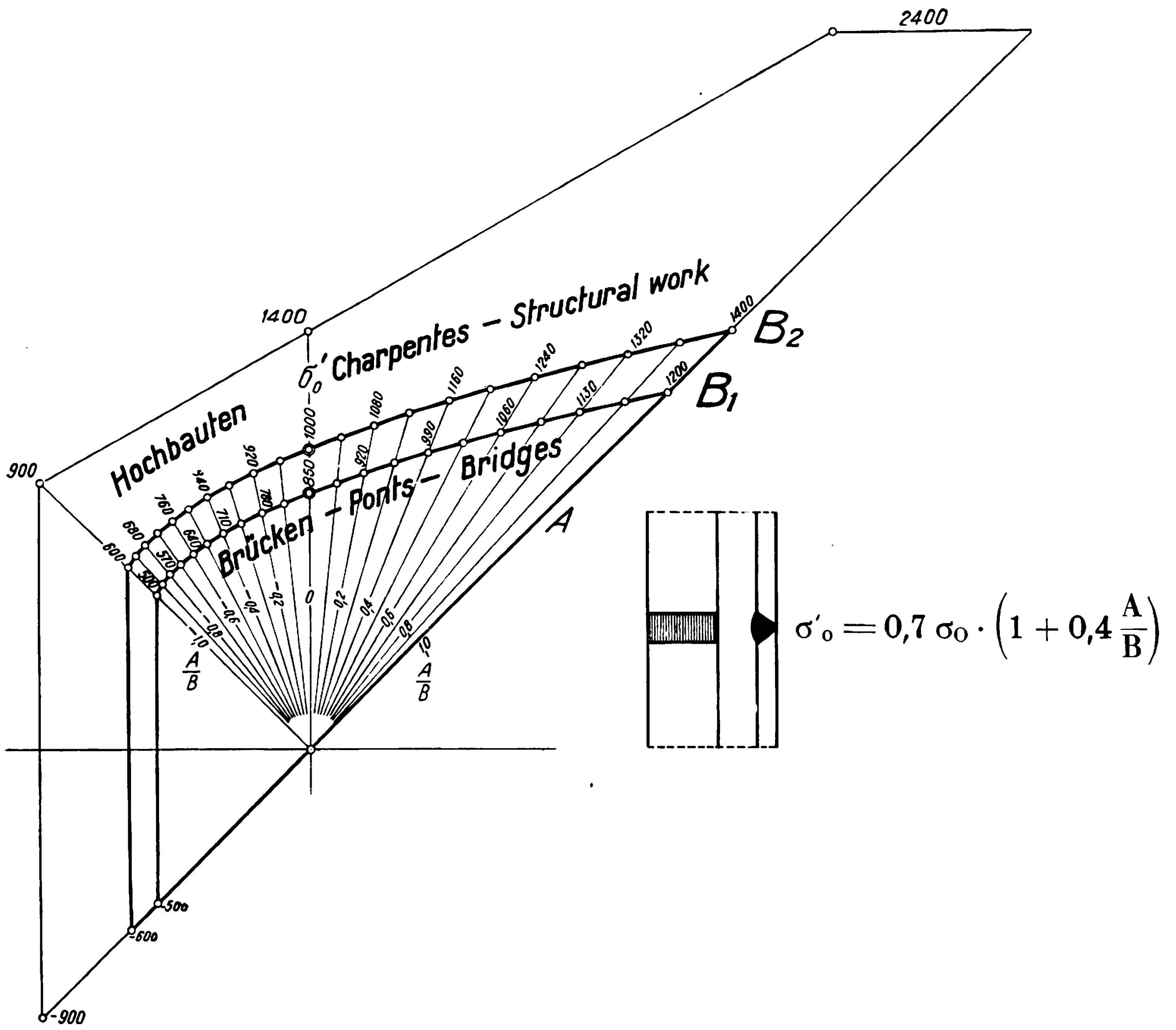

Fig. 16.

Butt weld — Permissible stresses. A = minimum, B = maximum.

Case 4.

The normal stresses σ_1 and σ_2 have opposite signs. The following is the comparison stress applies[9]:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \left(\frac{\sigma_2}{\alpha_2}\right)^2 + \beta \cdot \left(\frac{\sigma_1 \cdot \sigma_2}{\alpha_1 \cdot \alpha_2}\right) + 8 \cdot \xi^2} \leqq \sigma_{o\,zul} \qquad (7)$$

(Fig. 20)

The following are the figures for tension and compression:

	Tension	Compression	
$\alpha_1 =$	0.35	0.5	— Table 4 —
$\sigma_2 =$	0.85	1.0	
$\beta =$	$^1/_4$		
$\gamma =$	6		

Full clarity and sufficient experience has not yet been gained about the magnitude of the maximum permissible stress, and the behaviour of fillet welds under simultaneous longitudinal and shear stressing (e. g. fillet welds between web plate and flange plate of welded plate girders under bending).

It is absolutely indispensable that tests are carried out to determine the extent of the relieving action of the parent metal adjecent to the weld.

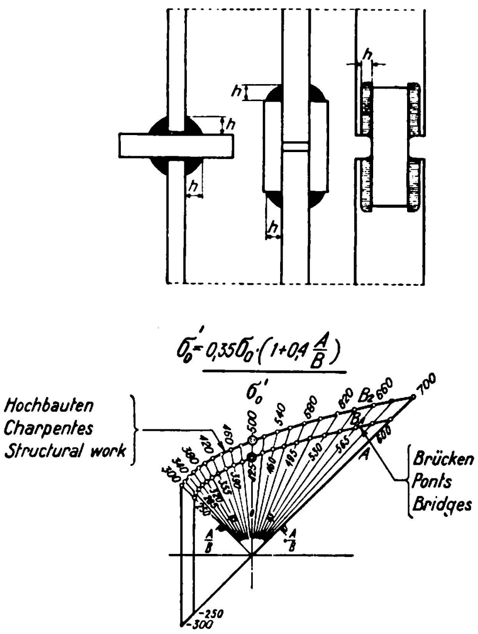

Fig. 17.

Fillet, side fillet and butt welds — Permissible stresses.

A = minimum
B = maximum.

Zone of Penetration.

Cases 5 and 6. — Butt-weld.

If the comparatively small difference between $\alpha_1 = 0.8$ instead of $\alpha_1 = 0.7$ is not taken into account then the ratios for the butt-weld itself can be used. — Cases 1 and 2, Fig. 19.

Case 7. — Fillet-weld.

For σ_1 and σ_2, both tension or compressive stresses, the following terms have to be used to express σ_g:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \gamma \cdot \tau^2} \lesseqgtr \sigma_{o\,zul} \tag{8}$$

and

$$\sigma_g = \sqrt{\left(\frac{\sigma_2}{\alpha_2}\right)^2 + \gamma \cdot \tau^2} \lesseqgtr \sigma_{o\,zul} \tag{9}$$

(Fig. 21)

Case 8. — Fillet-weld.

Are σ_1 and σ_2 of opposite signs, then σ_g is expressed by[9]:

$$\sigma_g = \sqrt{\left(\frac{\sigma_1}{\alpha_1}\right)^2 + \left(\frac{\sigma_2}{\alpha_2}\right)^2 + \frac{1}{3}\left(\frac{\sigma_1 \cdot \sigma_2}{\alpha_1 \cdot \alpha_2}\right) + \gamma \cdot \tau^2} \lesseqgtr \sigma_{o\,zul} \tag{10}$$

(Fig. 21)

The following are the figures for tension and compression:

	Tension	Compression	
$\alpha_1 =$	0.6	0.9	— Table 7 —
$\alpha_2 =$	0.85	1.0	
$\beta =$	$^1/_3$		
$\gamma =$	3		

The advantages of the mode of calculation as used by the E.M.P.A. are explained by the following examples.

Oblique butt-weld — Main stress σ_l, uni-axial.

The most favourable position of the joint from the practical point of view is that under 45° to the direction of the forces, in which case:

$$\sigma_1 = \sigma_2 = \tau = \frac{\sigma_l}{2}$$

and hence according to equation (5)

$$\sigma_l \sqrt{\left(\frac{0{,}5}{0{,}7}\right)^2 + 3 \cdot 0{,}5^2} = 1{,}12\, \sigma_l \leqq \sigma_{o\,zul}$$

$$\sigma_l = 0.89\, \sigma_{o\,zul}.$$

The advantage as compared to a butt-weld at right angles to the direction of forces:

$\frac{0{,}89}{0{,}70} \cong 1.275$ approximately 28 per cent. The gains are:

for the joint under . . .	30°	45°	60°
Amount of gain:	8 %	28 %	23 %

to the line at right angle to the direction of forces

Obliquely placed fillet-weld — Main stress $\frac{P}{h}$, uniaxial.

The most favourable placing of the weld joint from a practical point of view is done under 60° to the perpendicular line to the direction of forces P per unit length. Accordingly we receive

$$\sigma_h = \frac{P}{h} : - \sigma_1 = 0.25\ \sigma_h ; - \sigma_2 = 0.75\ \sigma_h ; - \tau = 0.433\ \sigma_h$$

$$\alpha_1 = 0.35 \qquad - \alpha_2 = 0.85$$

h = depth of weld.

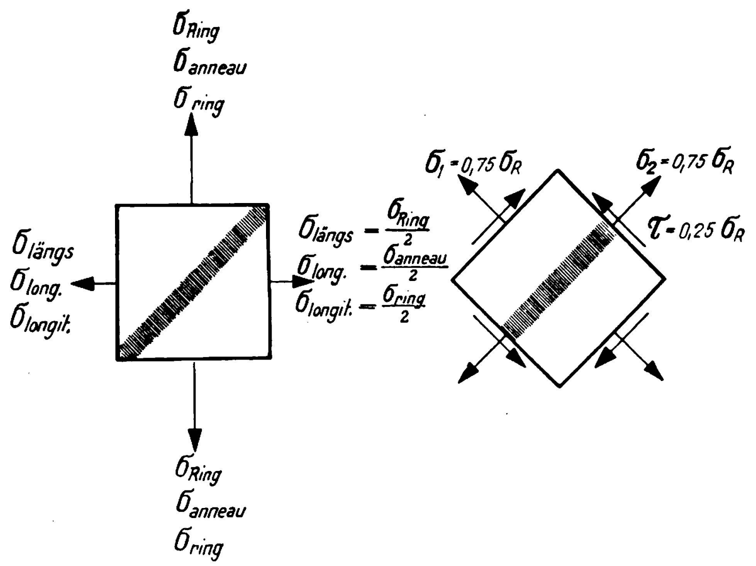

Fig. 18.
Helical weld — System of stresses.

From the equation (6) — Fig. 20 — follows that

$$\sigma_h \cdot \sqrt{\left(\frac{0,25}{0,85}\right)^2 + 6 \cdot 0,433^2} = 1,38\ \sigma_h \leq \sigma_{o\,zul}$$

$$\sigma_h \leqq 0.72\ \sigma_{o\,zul}$$

as compared to $\sigma_h \leq 0.35\ \sigma_{o\,zul}$ for a fillet weld which is at right angles to the direction of force, the gain is 100 per cent.

Helical welds of containers, boilers and pressure pipes.

The stress conditions of a two-axially stressed weld (the third main stress which is equivalent to the internal pressure for the member on to the inside face of an element can be disregarded on account of its insignificant amount) is shown in Fig. 18. The application of the relation according to formula (5) results in:

$$\sqrt{\left(\frac{\sigma_1}{0,7}\right)^2 + 3\,\tau^2} \leqq \sigma_{o\,zul}$$

$$\sqrt{\left(\frac{0,75\ \sigma_{Ring}}{0,70}\right)^2 + 3 \cdot (0,25\ \sigma_{Ring})^2} = 1,15\ \sigma_{Ring} \leqq \sigma_o$$

hence we receive

$$\sigma_{Ring\,zul} = 0.87\ \sigma_{o\,zul}$$

(Ring = annular, ring [ring stress])

Compared to alongitudinal joint running parallel to the axis of the cylinder, and for $\sigma_{\text{Ring zul}} = 0.7\ \sigma_{\text{o zul}}$ the helical weld offers a gain of: $\frac{0{,}87}{0{,}70} = 1.25$, i. e. equal to 25 %. For a butt-weld which has been freed from stresses in an annealing over[11], the permissible annular stress can be put: $\sigma_{\text{Ring zul}} = 0.8\ \sigma_{\text{o zul}}$ so that helically welded containers annealed as a whole become equal in strength as seamles containers ($\sigma_{\text{o zul}}$ in the case of riveting)[12].

Through appropriate disposition and design of weld seams in very great advantages are obtainable in favour of welded structures.

For *multi-axial stressing due to alternating load effects* (fatigue), for the whole range of oscillating loading effects up to the yield limit, the permissible stresses (comparison stresses) for butt-welds, fillet welds and the zones of penetration can be taken in accordance with the data given in Table 3 — Figs. 15, 16 and 17 — whereby the relations expressed by the formulae 2 to 10 — Figs. 19, 20 and 21 — must be considered.

VI. Experience.

The branch with the longest experience is that of *container and recipient construction;* it has fulfilled all expectations if correctly designed and executed in workmanlike manner.

The construction of welded *pressure pipes* is of later date and apart with a few ill successes has proved satisfactory.

Recently the pressure piping construction has adopted the use of *steels with a higher carbon content* and heavier gauge walls; sufficient experience in this line is still lacking, provided however, that the work is treated correctly from the metallurgical point of view, it promises to become a full success.

With regard to the welding of high carbon or alloy steels, there are both successes and failures to report. The fatigue strength of steels with higher carbon content, with maximum carbon content and alloyed steels is not very much higher, or may be not at all higher than that of standard structural mild steel; or only little higher in case of slight pre-stressing. Only for strong pre-stressing the fatigue strength becomes greater than for standard structural steel, and it is only under these conditions that the advantages of high carbon steel become more marked.

When selecting high carbon steel for the purpose of welded structures it is necessary to exercise great care. Special precautionary meassures should be taken.

Thomas steel which has been produced under perfect conditions from the point of view of technical properties can be welded just as satisfactorily as *Siemens-Martin-steel* as prescribed by regulations; the welds are practically equal from the point of view of strength and deformation.

Structural engineering is now about to adapt itself to the special methods requisite for welding, the properties of strength and deformation of welds as far as these affect the design and detail of construction.

---

[11] Stress-free annealing of fillet welds improves the strength of the zone of penetration, but not the fillet weld itself.

[12] For fully seamless containers the fatigue strength is higher than for riveted containers, so that seamless containers are superior than helically welded ones.

Bridge engineering is still in its infancy; in particular precaution has still to be exercised with lattice bridge constructions. Conditions are less complicated with plated constructions (girders, frames, arches).

The problems with welded lattice constructions are not yet solved particularly as regards the connection of members. The points of intersection of members

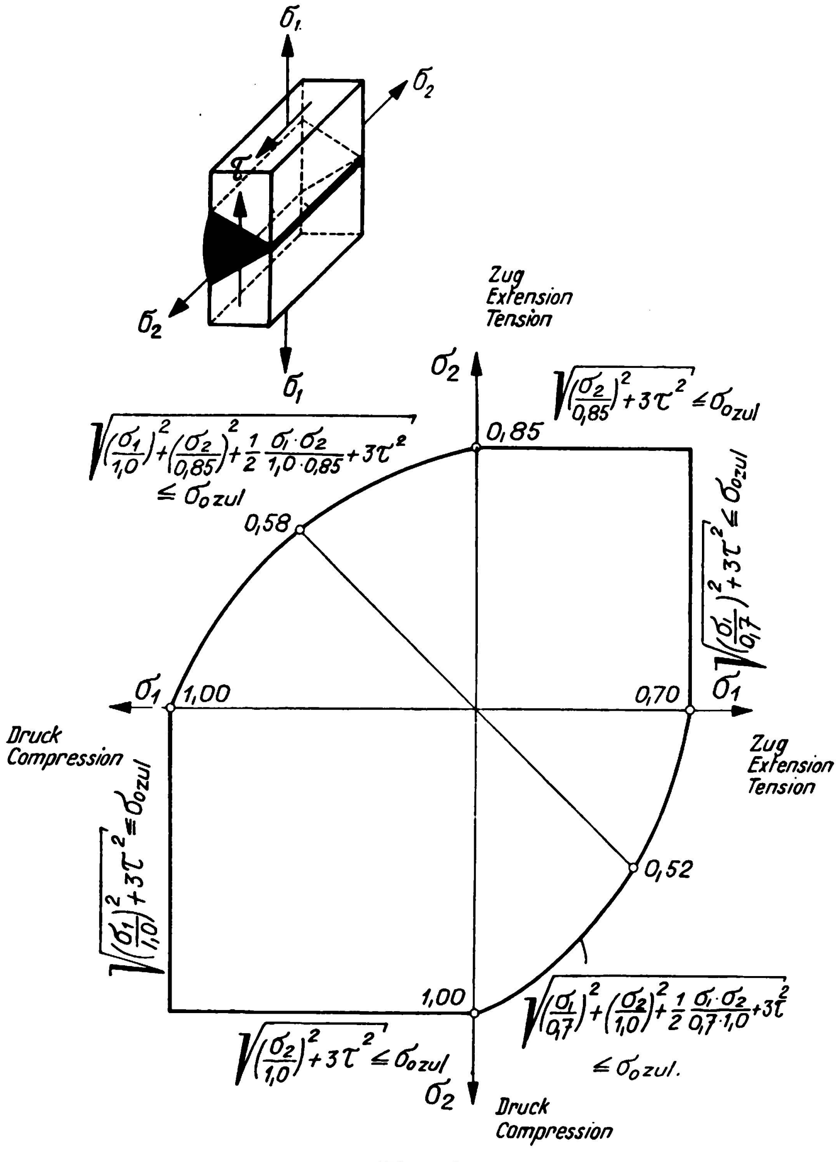

Fig. 19.

Butt welds, two-axially stresses — Permissible stressing conditions.

(panel points) show very little resiliency (equalisation of stresses), compared with riveted constructions. Higher secondary stresses, due to less flexibility of the panel point connections, and higher values of local stress accumulations on account of sudden force transmissions and high additional stresses which cannot be calculated, due to thermal influences during welding (contraction), lead often to premature fatigue effects (cracks, fractures).

The use of electrodes which are not too thick and the execution of welding by layers which individually are not too heavy should be given preference on account of better quality of welds in place of thick electrodes and heavy passes, where due to rapid cooling a brittle texture of the weld metal (Widmannstädt texture) is created.

This brittle texture can be made to disappear only by heating the metal to a temperature above the upper of transformation temperature. This is possible only for the inner layers, but the outer ones can only be treated by subsequent annealing with the burner or in an annealing oven.

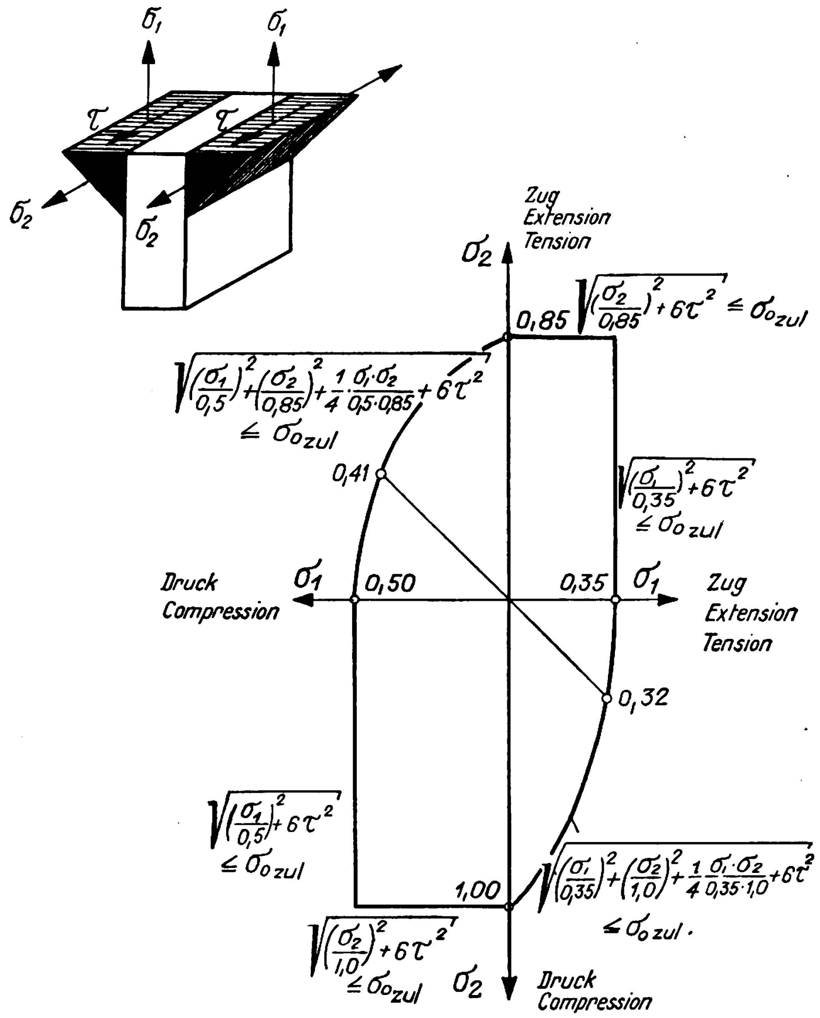

Fig. 20.

Fillet, side fillet and butt welds, two-axially stressed — Permissible stressing conditions.

Internal stresses are dangerous to welded structures only if the assemblage of the various parts is not properly carried out, or if the various members are too rigid (do not give) so that fine cracks begin to form or finally, if the internal stresses become too great.

If the method of manufacture or nature of construction and therefore, of execution, allow, the welded joints should be heated or annealed with the burner at a temperature above that of the upper transformation temperature, with subsequent annealing of the whole piece to a temperature reasonably below the lower temperature of transformation. In this way it is possible to normalise the outer weld layers which are too brittle on account of rapid cooling and to render them more ductile again. This process on the other hand allows also to eliminate or at least to reduce the internal stresses. Stress-free annealing is to be recommended, wherever possible.

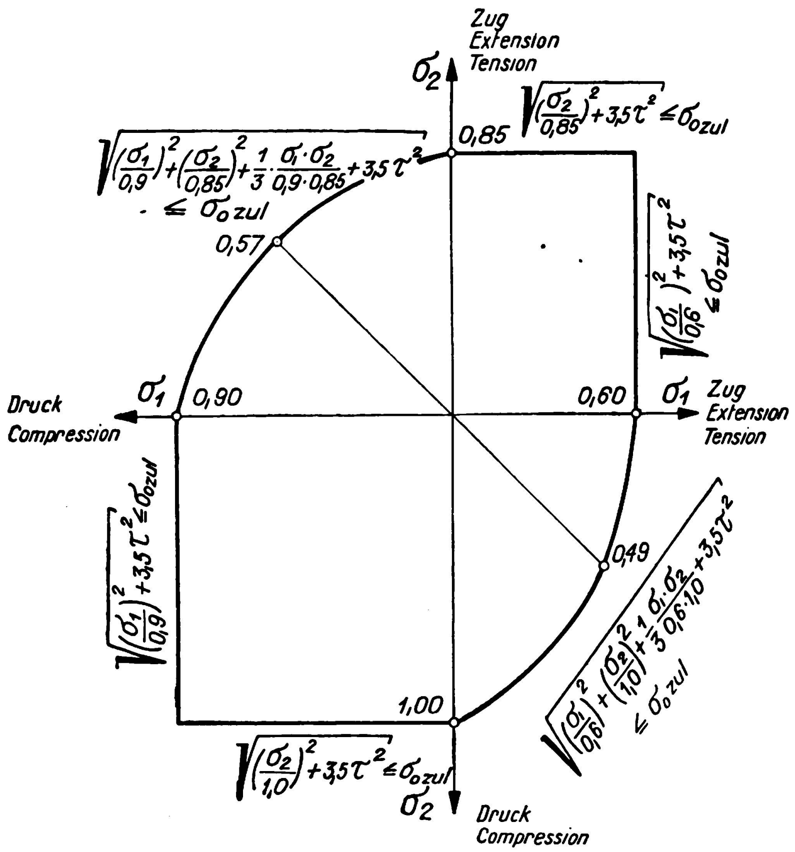

Fig. 21.

Penetration zones of fillet, side fillet and butt welds, two-axially stressed — Permissible stressing conditions.

The following are some of the causes which are known to have been responsible for *failures*:

The use of unsuitable, high carbon, and impure steel with blow holes squashed by rolling (so-called straw texture, flaws, etc.);

defective mechanical treatment — forcible stretching and cold bending of rigid sections;

unsuitable thermal treatment — ommision to preheating, heating of cold-stretched steels in the zone of re-crystallisation;

application of a flame which of insufficient reducing power and the use of unsuitable, non-protective and non-reducing electrodes;

un-workmanlike constructional treatment — notching with the oxy-acetylene burner, sudden variation of cross sections;

concentration of stresses and excessive and real fatigue stresses.

Summary.

The new Swiss Federal Regulations relating to the calculation, the construction and the upkeep of steel structures under the control of the Confederation include provisions concerning the *strength and properties of deformation* (static tensile strength and bending coefficient), and *permissible stresses* which are based on the static and fatigue tests on welded connections which were carried out by the Swiss Federal Institute for the Testing of materials (E.M.P.A.) at Zurich in the years 1927 to 1935.

The permissible stresses are based on the results of fatigue strengths tests which characterise the technical value of welded joints more clearly and more accurately than the static resistances.

The *safety factor* adopted for welded connections is of practically the same order as for riveted connections.

The method of calculation which is used by the Swiss Federal Institute for the Testing of Materials for uni- and multi-axial stresses, is based on the deformation hypothesis and takes into account the strength ratios of welded and riveted connections. It is a method which will allow the further development of workmanlike and properly designed welded connections.

The recognition given to *research work* in connection with the *technical aspect of resistance* and *experience* have been very successful in developing electrodes and the welding of high-grade steel, the general arrangement and structural design of detail, measures governing execution, methods of calculation and the control and inspection of welded structures.

IIIa3

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

Einfluß der Gestalt der Schweißverbindung auf ihre Widerstandsfähigkeit.

Influence de la forme des assemblages soudés sur leur résistance.

O. Graf,
Professor an der Technischen Hochschule Stuttgart.

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

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

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

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

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

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

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

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

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

a) Re question No 1.—

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

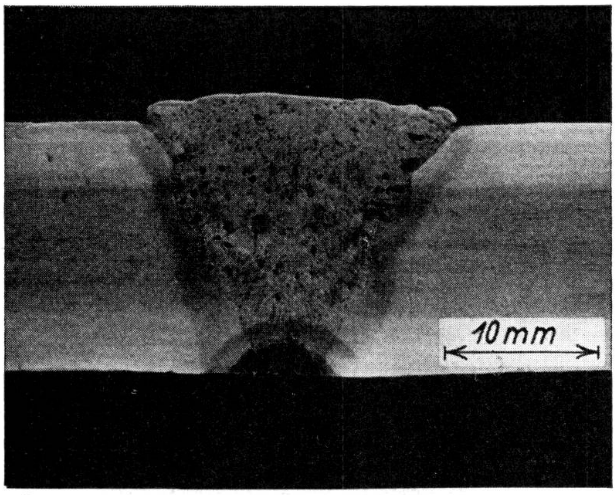

Fig. 1.

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

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

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

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

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

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

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

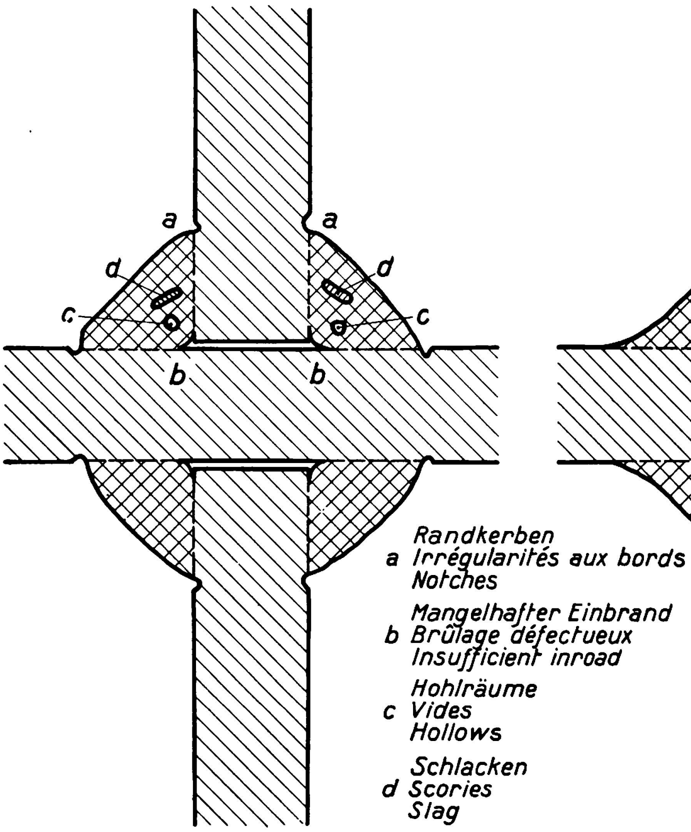

Fig. 2.

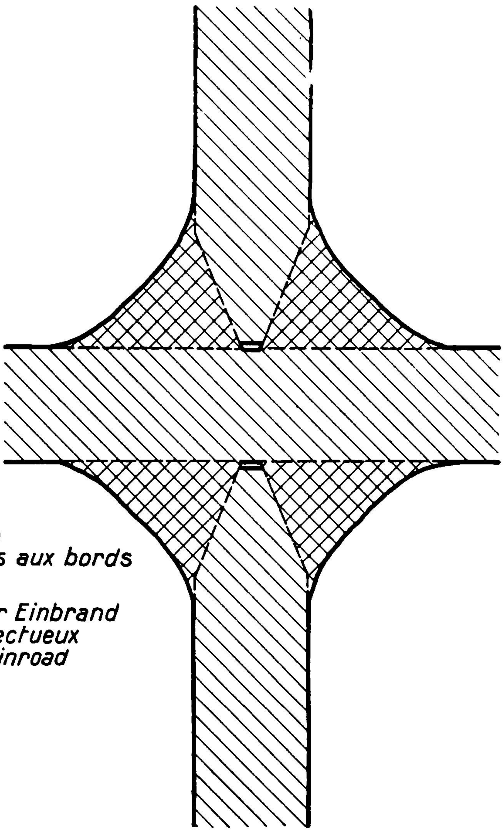

Fig. 3.

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

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

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

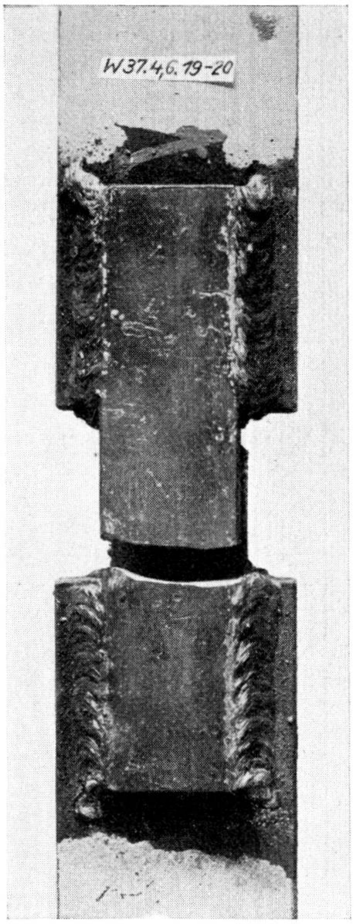

Fig. 4.

Side fillet weld connections after fatigue test.

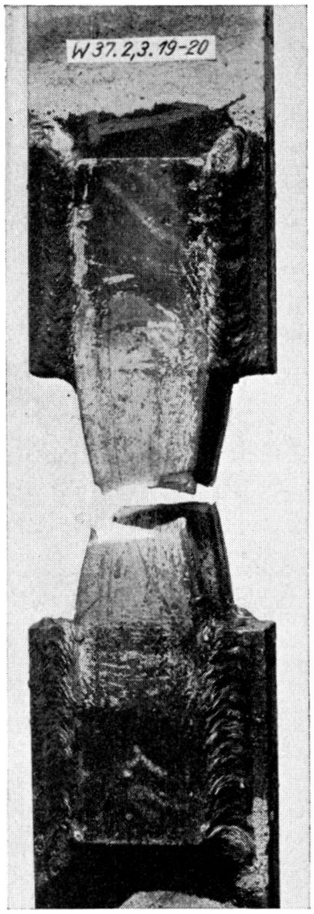

Fig. 5.

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

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

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

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

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

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

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

Fig. 6 is an example of the results obtained from tests on a butt-weld. The strength to withstand the stationary and the total loading is defined by the tensile strength and the yield point. The stress amplitude which the material can stand for frequently repeated loads, has been ascertained as $S = 14{,}5$ kg/mm² based on surge load strength and $S = 13{,}1$ kg/mm² for the yield point. This enables the permissible stress to be selected for the static load and the total load, and independently of them, the permissible stress for the frequently recurring load. If, for instance, the permissible stress has been fixed at 0,8 times the ascertained strength, then, in the case of Fig. 6, the permissible stresses would work out at:

a) For static loads and for the total loads: $37.8 \cdot 0.8 =$ approx. 30 kg/mm² and

b) For frequently recurring loads: $14.5 \cdot 0.8 =$ approx. 11 kg/mm².

The maximum permissible stress would therefore be 30 kg/mm². Of this maximum stress, 11 kg/mm² may be set up by live loads.

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

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

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

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

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

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

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

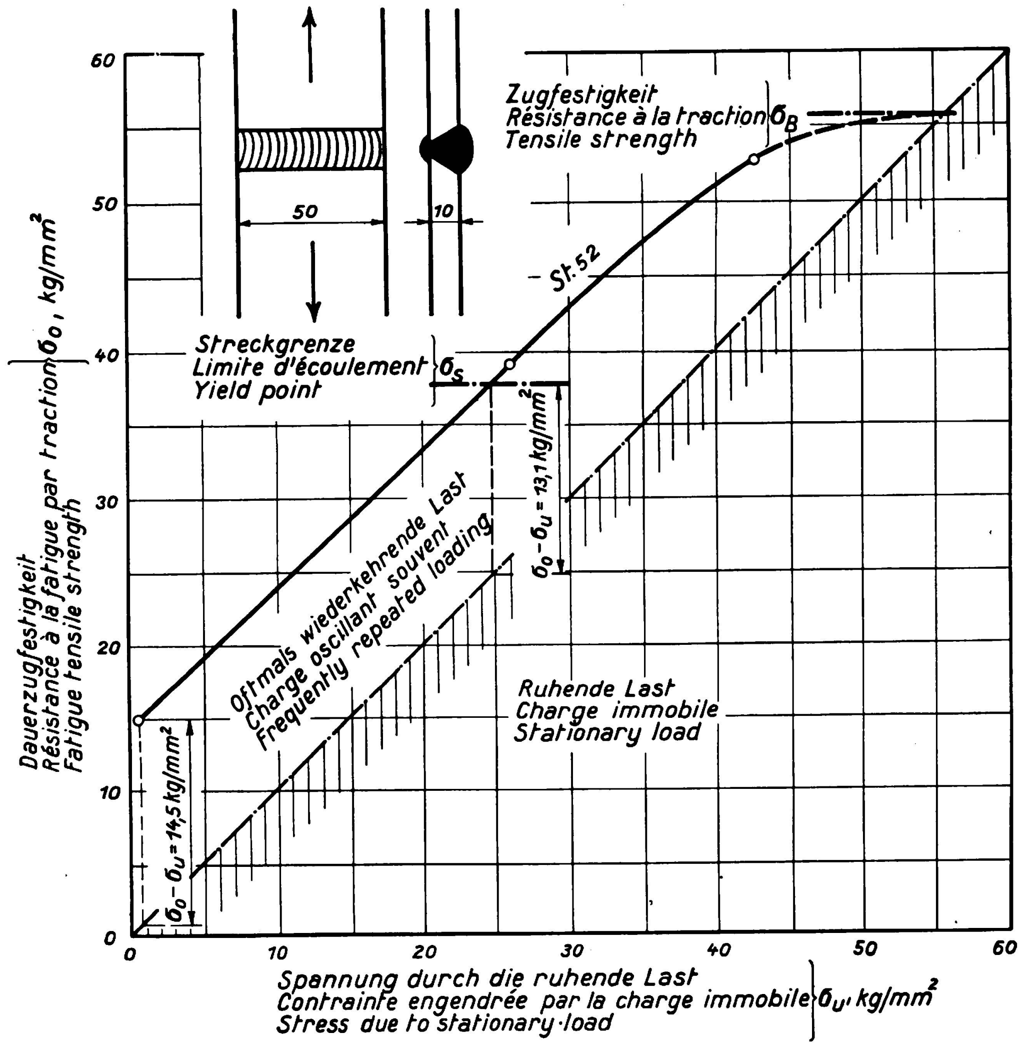

Fig. 6.

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

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

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

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

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

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

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

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

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

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

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12 „Bautechnik" 1932, p. 415.

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

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

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

tions — has gradually become smaller. Whereas the surge load tensile strength can be put at roughly 18 kg/mm² for good butt-welds, figures of up 12 kg/mm² may now be expected for properly designed and well made side-fillet welds as in Fig. 7. The carrying capacity given for the butt-weld can only be guaranteed where an X-ray examination has proved it to be satisfactory. X-raying is not necessary in the case of the side-fillet welded joint, provided it has been made by careful and skilled welders.

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

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

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

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

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

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

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

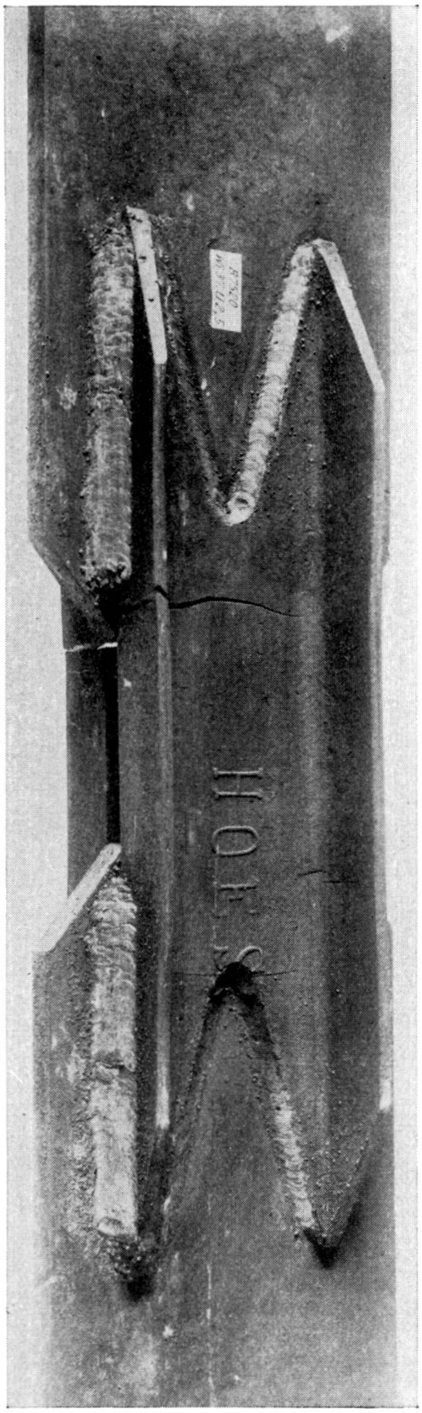

Fig. 7.

Fig. 8.

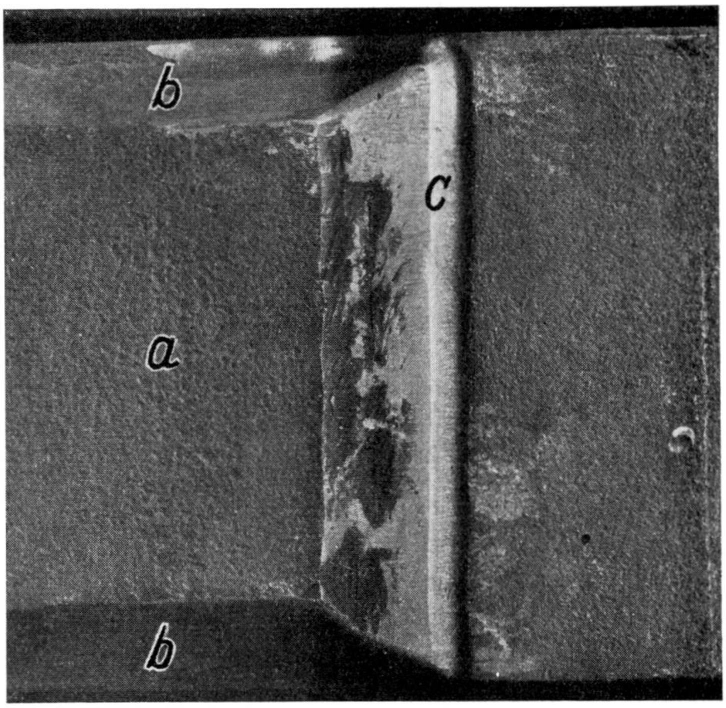

Fig. 9.

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

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

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

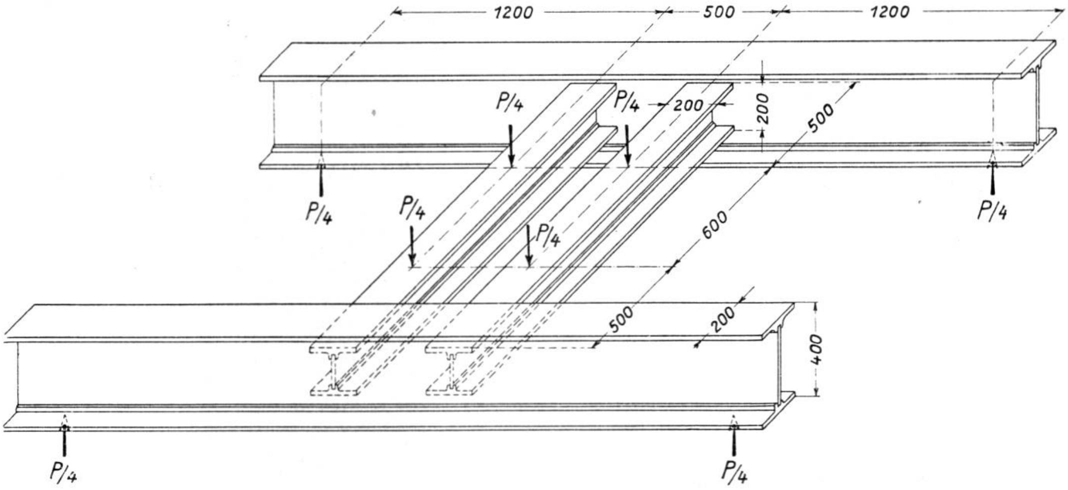

Fig. 10.
Tests with welded systems

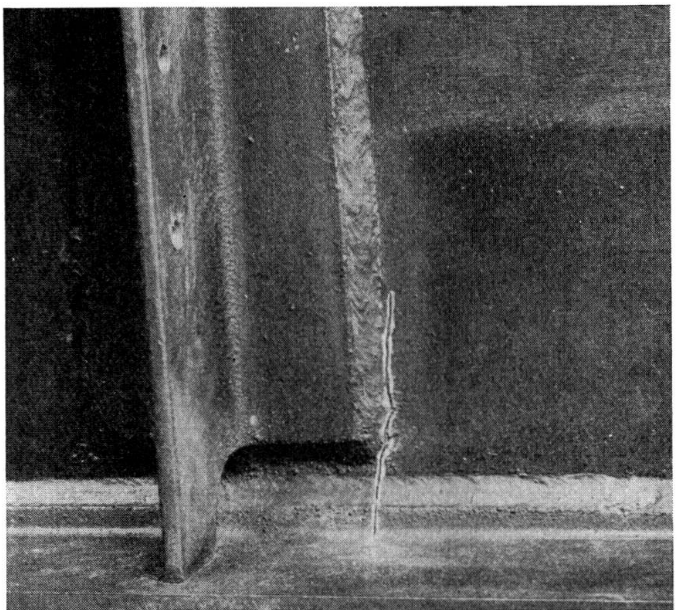

Fig. 11.

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

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

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

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

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

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

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

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

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

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

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

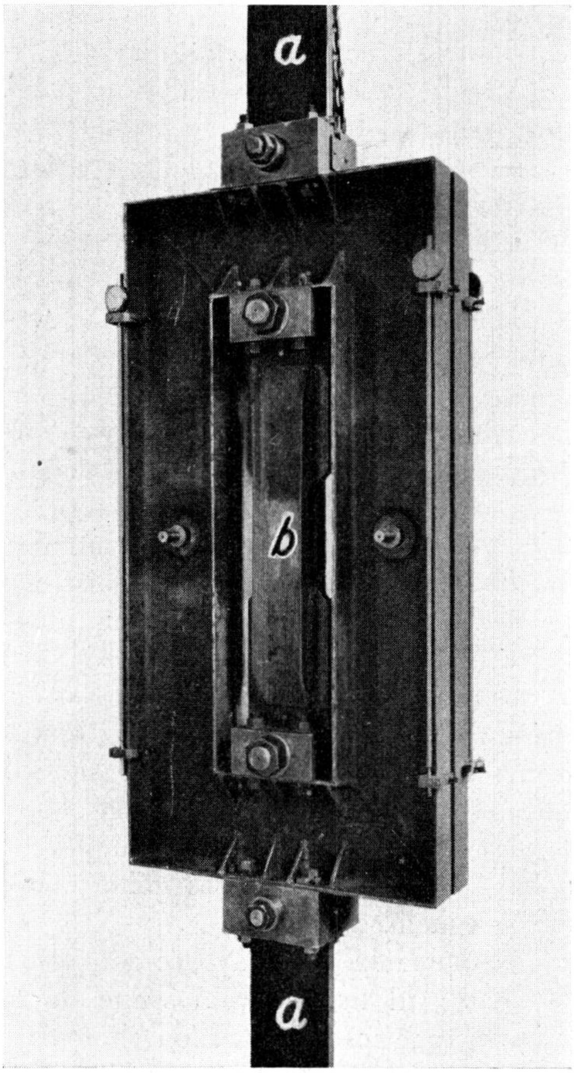

Fig. 12.

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

Summary.

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

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

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