The influence of frequently alternating loading on welded structures

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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 resiptance 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°. 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° 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.

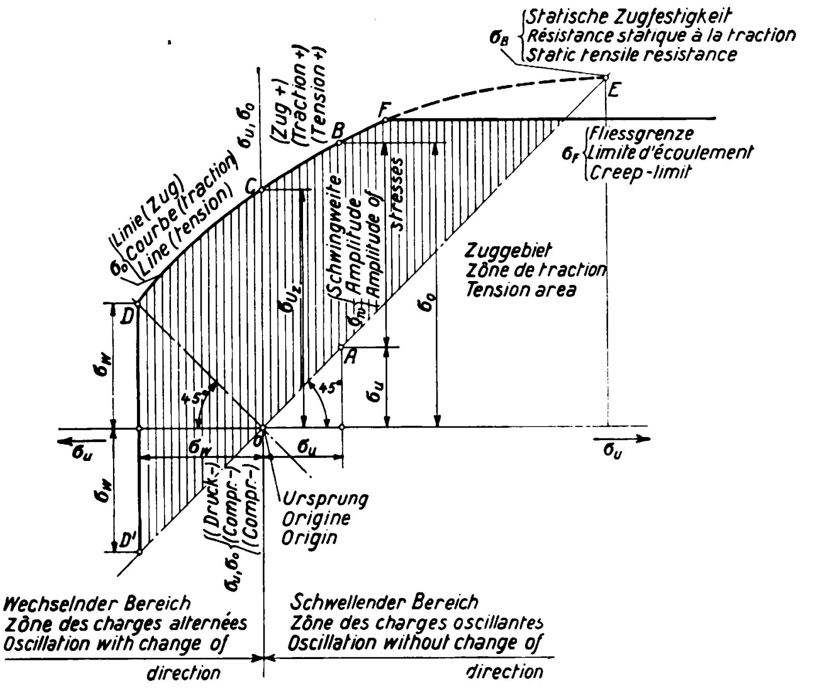


Fig. 1.

Curve of fatigue strength (σ_0-line) if σ_0 represents tension.

the vertical axis lies the range of alternating forces or oscillation of forces with change of direction, where the stresses σ_0 and σ_u have reverse signs.

The stresses σ_0 for the tensile area in fig. 1 can only go up to the yield point

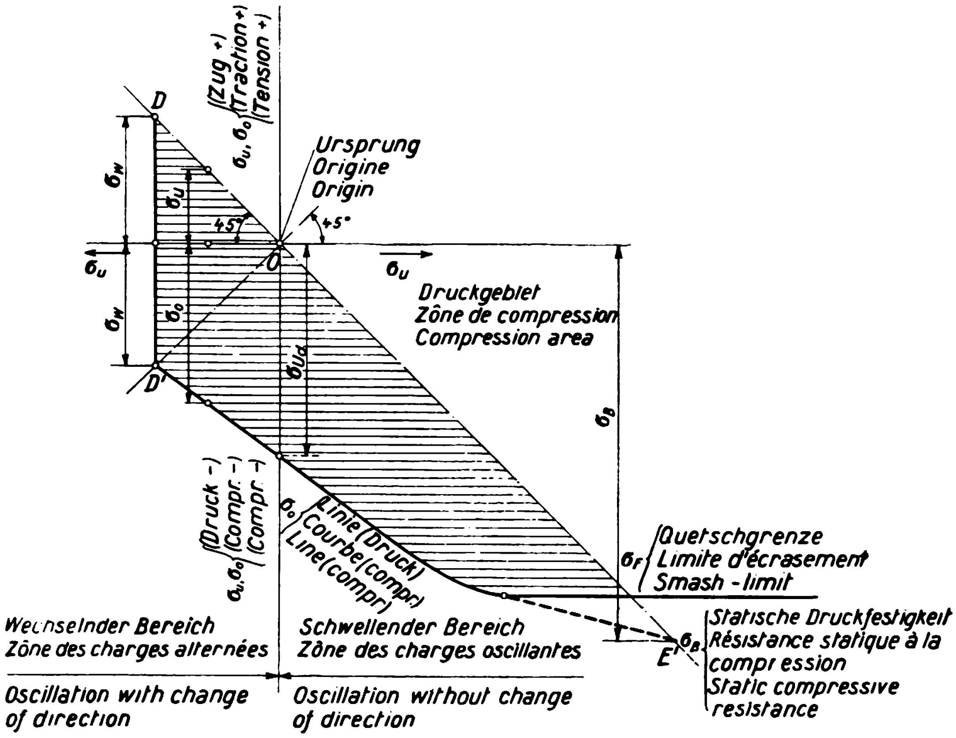


Fig. 2.

Curve of fatigue strength (σ_0-line) if σ_0 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.

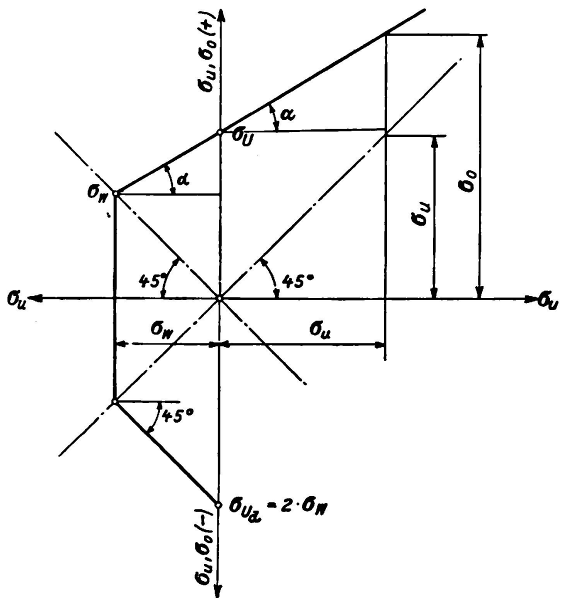


Fig. 3.

The angle α is expressed by the relation

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

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

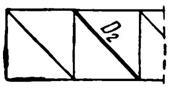
For the compressive zone we can assume with sufficient accuracy that the amplitude of oscillation σ_w has everywhere the same value $\sigma_w = 2\,\sigma_W$. This means that in this case the angle $\alpha = 45^0$. The surge load strength for the zone of compression has accordingly been laid down as $\sigma_{Ud} = \sigma_w = 2\,\sigma_w$. 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.

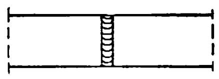
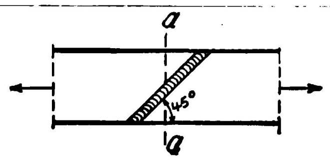
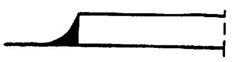
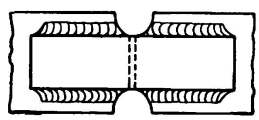
1	2	3	4	5	6	7
Span l m		member max stress in double track Railway Bridges in st. 37 I one track fully loaded, 2nd track not-loaded kg/cm²	II one track fully loaded, the other track loaded with 3,6 t/m kg/cm²	III one track fully loaded, the other track loaded with 8,0 t/m kg/cm²	IV one track fully loaded, 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
70	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
100	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_{U_3} 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/mm²) remarkably low endurance-strength (fatigue strength) values (surge load strength $\sigma_U = 8$ kg/mm²), 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.

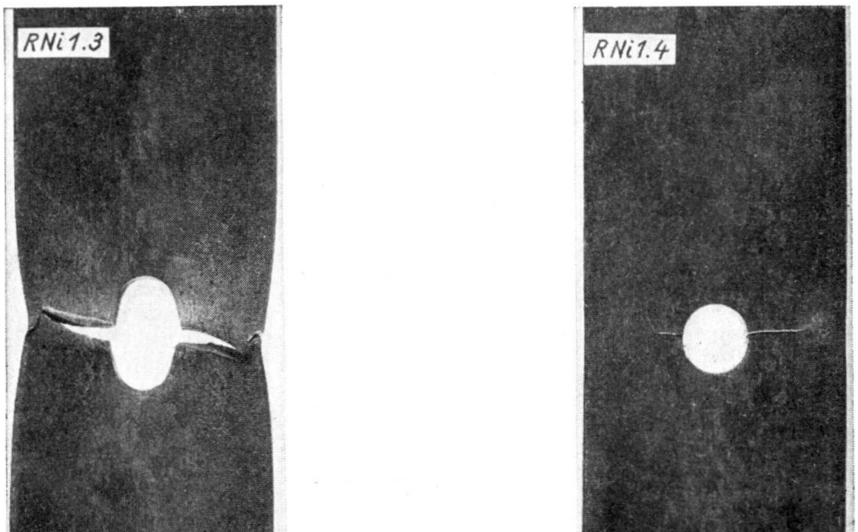


Fig. 4.

Static tensile test
tensile strength
$\sigma_B = 54.6$ kg/mm²

Fatigue tensile test
surge load strength
$\sigma_U = 24.0$ kg/mm²

Trials with perforated test bars.

[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/mm² whilst the dynamically stressed piece had a surge-load strength of $\sigma_B = 24$ kg/mm² 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/mm² as high as for a pierced-flat.

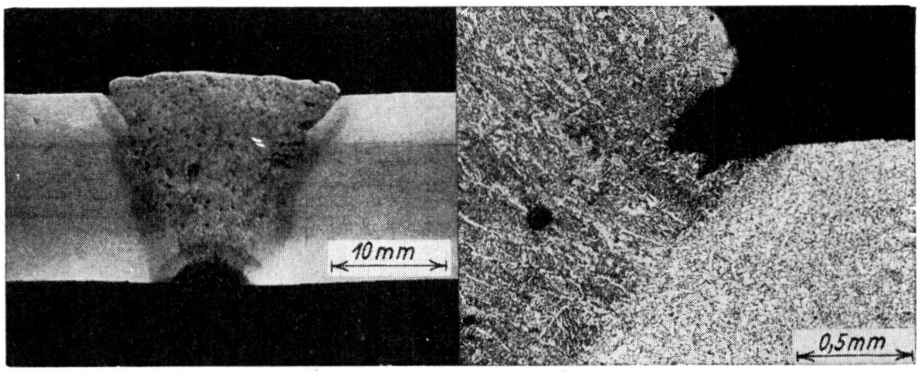


Fig. 5a.

Surge load strength $\rho_{Uz} = 10$ kg/mm²
Static tensile strength $\sigma_B = 34$ kg/mm².

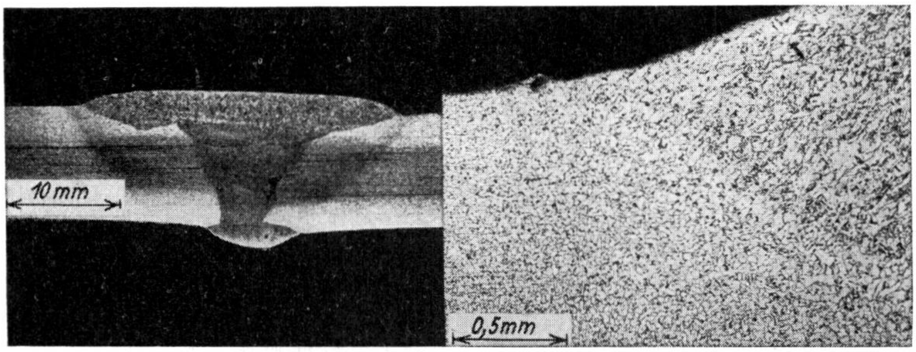


Fig. 5b.

Surge load strength $\rho_{Uz} = 18$ kg/mm².
Static tensile strength $\sigma_B = 37.5$ kg/mm².
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

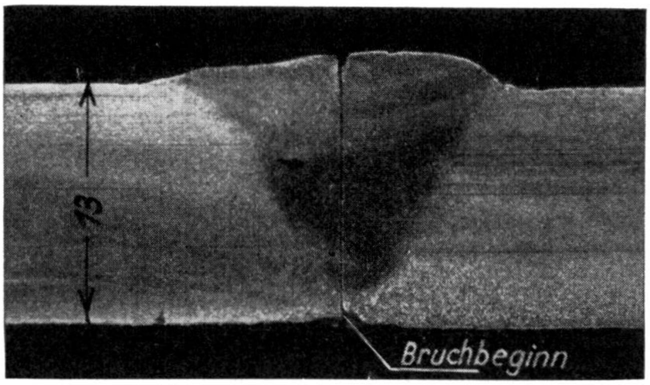


Fig. 6a.

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

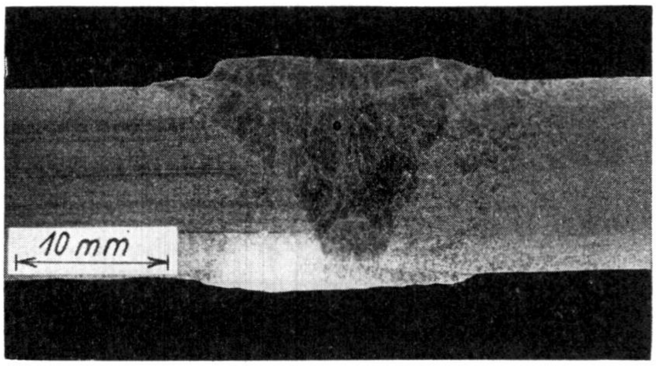


Fig. 6b.

Surge load strength $\rho_{Uz} = 18$ 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 $\sigma_U = 24\ \mathrm{kg/mm^2}$ 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.

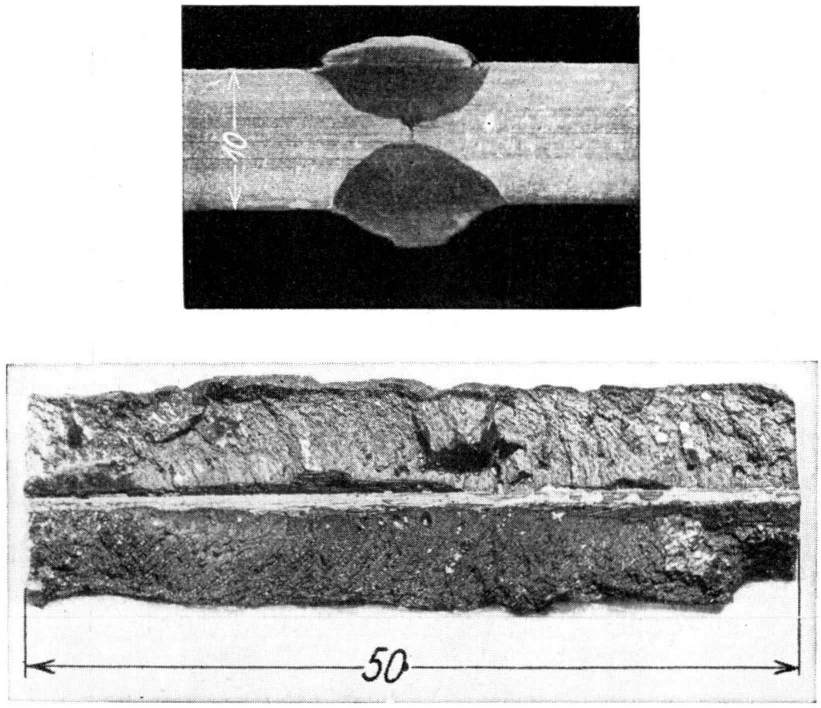


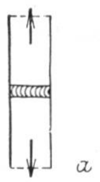
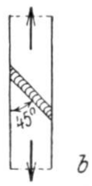
Fig. 7.

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 45°. 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

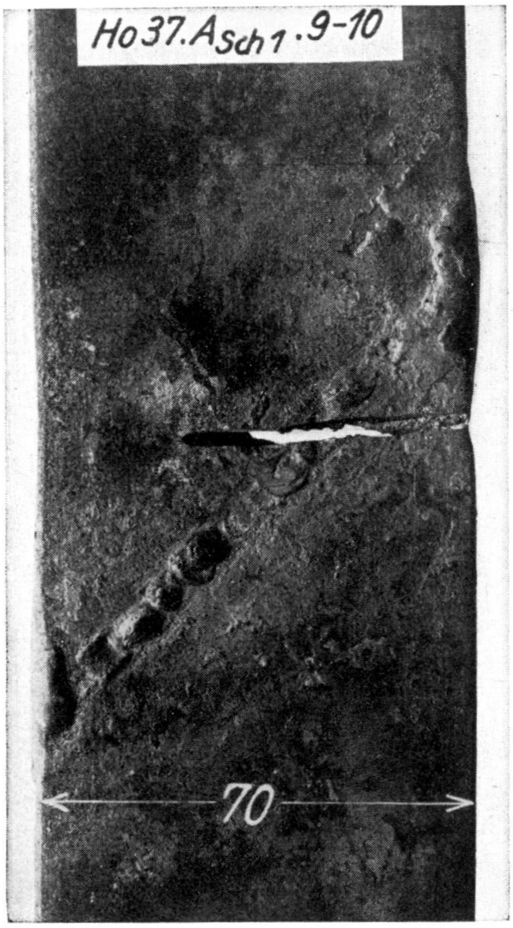


Fig. 8.
(root not rewelded.)

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/mm². For butt-welds with rewelded roots and dense, compact weld metal, however, the following surge-load strengths were obtained:

$\rho_U = 18$ kg/mm² for joint right angled to the direction of forces and

$\rho_U = 22$ km/mm² joint under 45⁰ 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/mm².

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/mm²

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

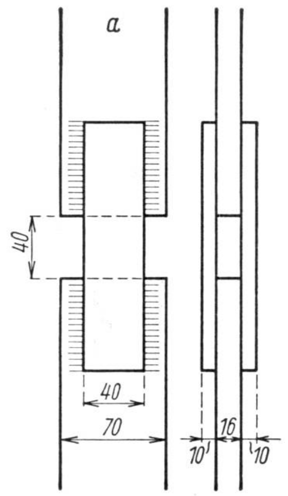


Fig. 10.

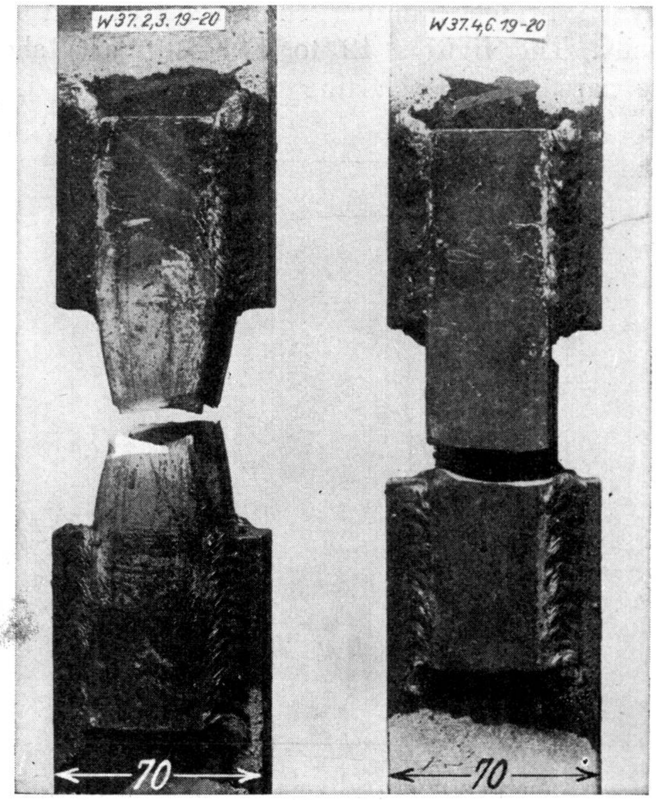


Fig. 11.

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

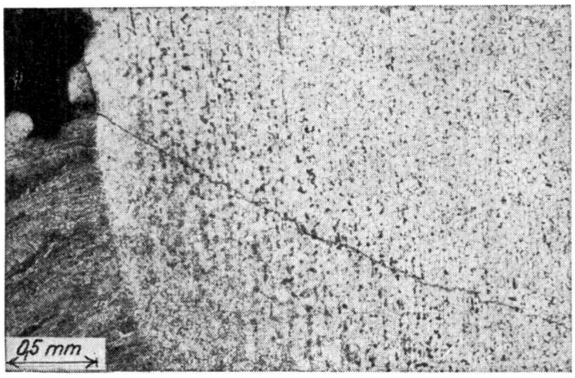


Fig. 12.

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).

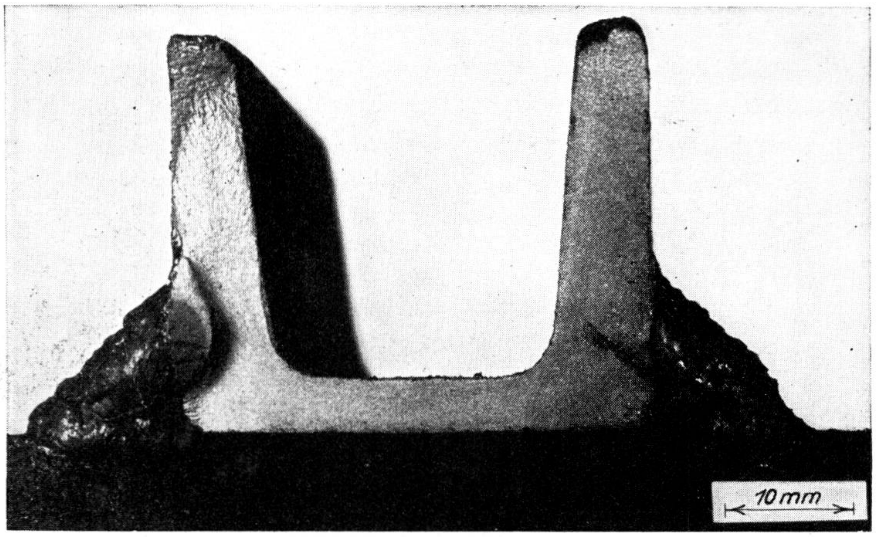


Fig. 13.

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-

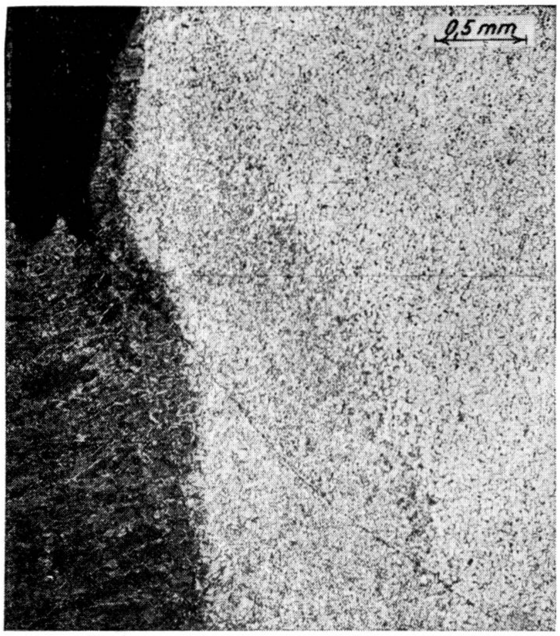


Fig. 14.

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/mm².

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/mm²	ρ_o kg/mm²	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/mm² with 2 kg/mm² pre-stressing

Amplitude of stress $\rho_w = 6{,}5$ kg/mm²

After machining weld ends (fig. 16) $\rho_o = 12{,}5$ kg/mm² with 2 kg/mm² pre-stressing

Amplitude of stress $\rho_w = 10{,}5$ kg/mm².

With a pre-stressing of 10 kg/mm² was found for untreated weld ends $\rho_o = 16$ kg/mm², hence an amplitude of oscillating stresses of $\rho_w = 6$ kg/mm². 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/mm² to $\rho_U = 11$ kg/mm² 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.

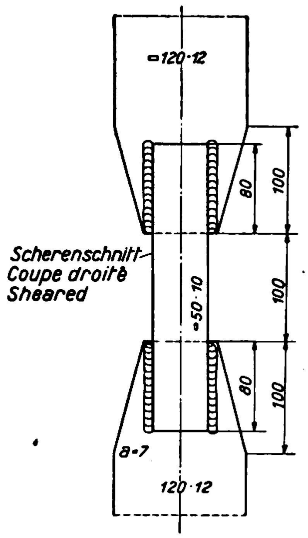


Fig. 15.

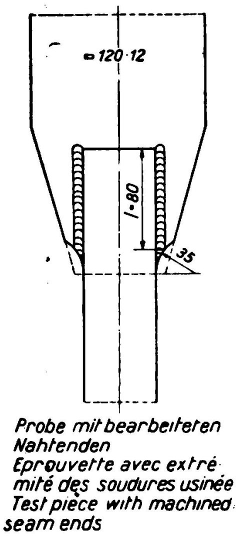


Fig. 16.

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.

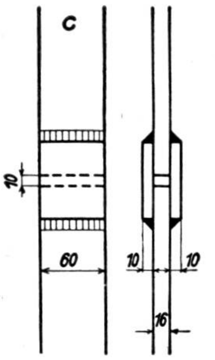
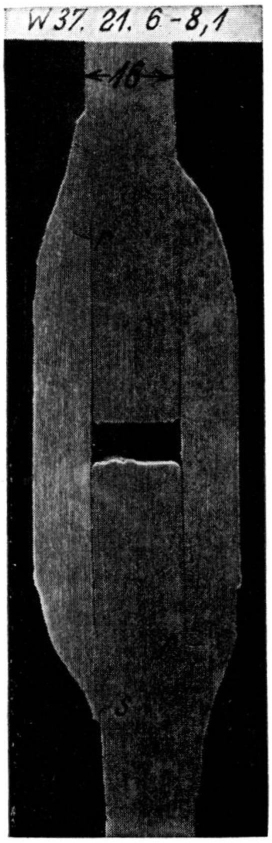


Fig. 17.

Fatigue tensile test with end fillet weld connections.



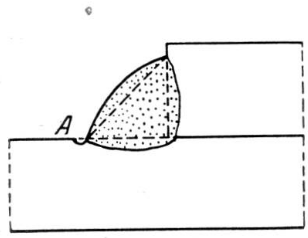


Fig. 19.

Fig. 18.

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

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.]

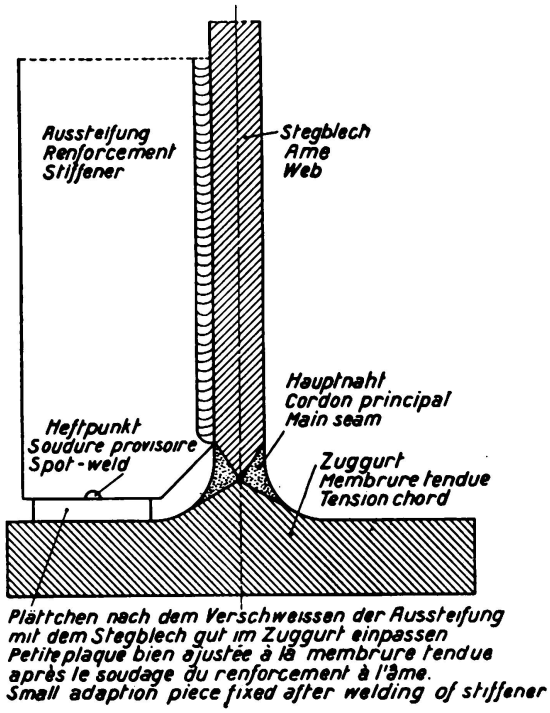


Fig. 20.

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.

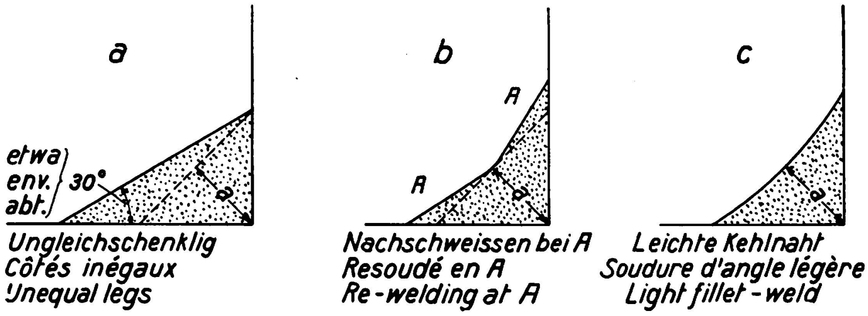


Fig. 21.

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 45⁰ 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 $\rho_U = 7$ kg/mm² for an arc welded end fillet under 45^0 with the value found in Dahlem of $\rho_U = 9{,}3$ kg/mm² 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 $\rho_U = 9{,}5$ kg/mm², 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.

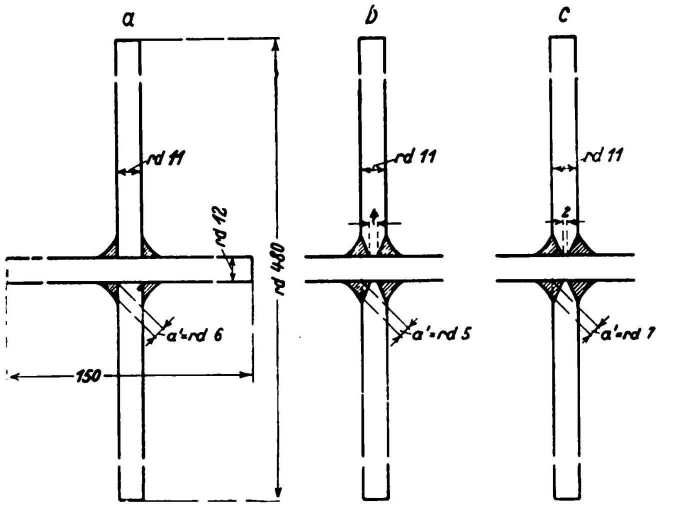


Fig. 22.

Welded cruciform connections (arc-welding).

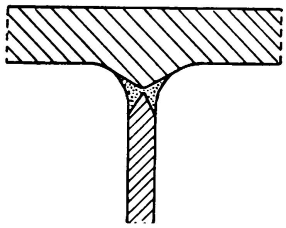


Fig. 23.

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

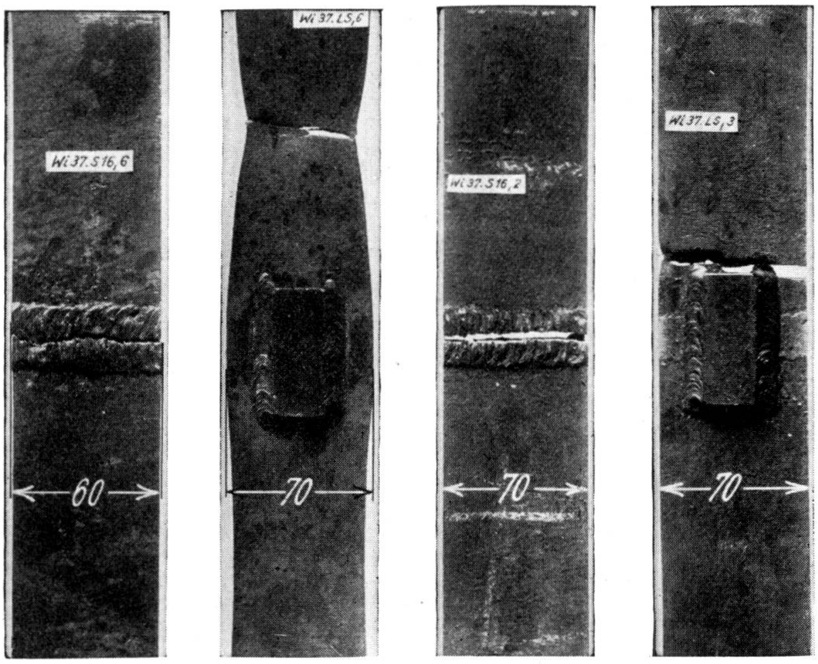


Fig. 24.

Static tensile tests Fatigue tensile tests.

σ_B 30.4 38.4 kg/mm² 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.

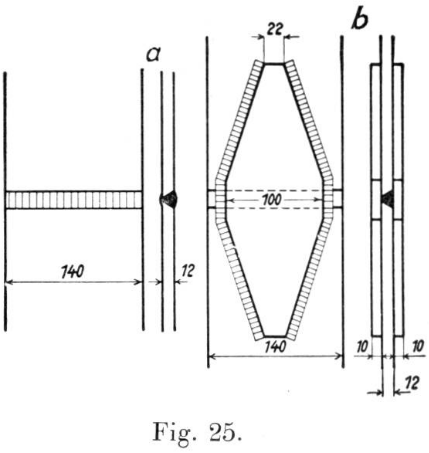


Fig. 25.

σ_U 13 10 kg/mm²

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

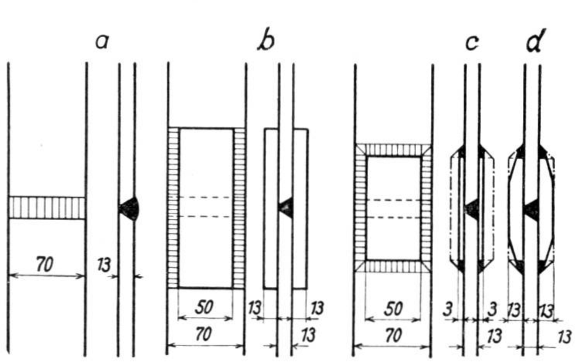


Fig. 26.

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 ($\sigma = 37$ to $42\ kg/mm^2$). 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:

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

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

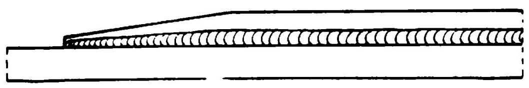


Fig. 27.

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).

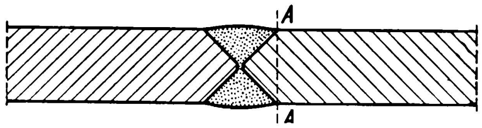


Fig. 28.

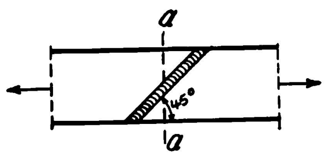


Fig. 29.
Butt weld under 45° of best workmanship.

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 $\sigma_U = 18\ kg/mm^2$ up to $\sigma_U = 22\ kg/mm^2$. 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 $10\ kg/mm^2$ (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/mm^2) 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/mm^2 and the surgeload strength between 8,0 and 22,0 kg/mm^2.

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 $\left\{\begin{matrix} 1400\ \text{kg/cm}^2 & \text{for st. 37} \\ 2100\ \text{kg/cm}^2 & \text{for st. 52} \end{matrix}\right\}$ 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

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

leading to the formula

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

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 $\sigma_{zul} = 1400$ 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

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

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 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

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

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.

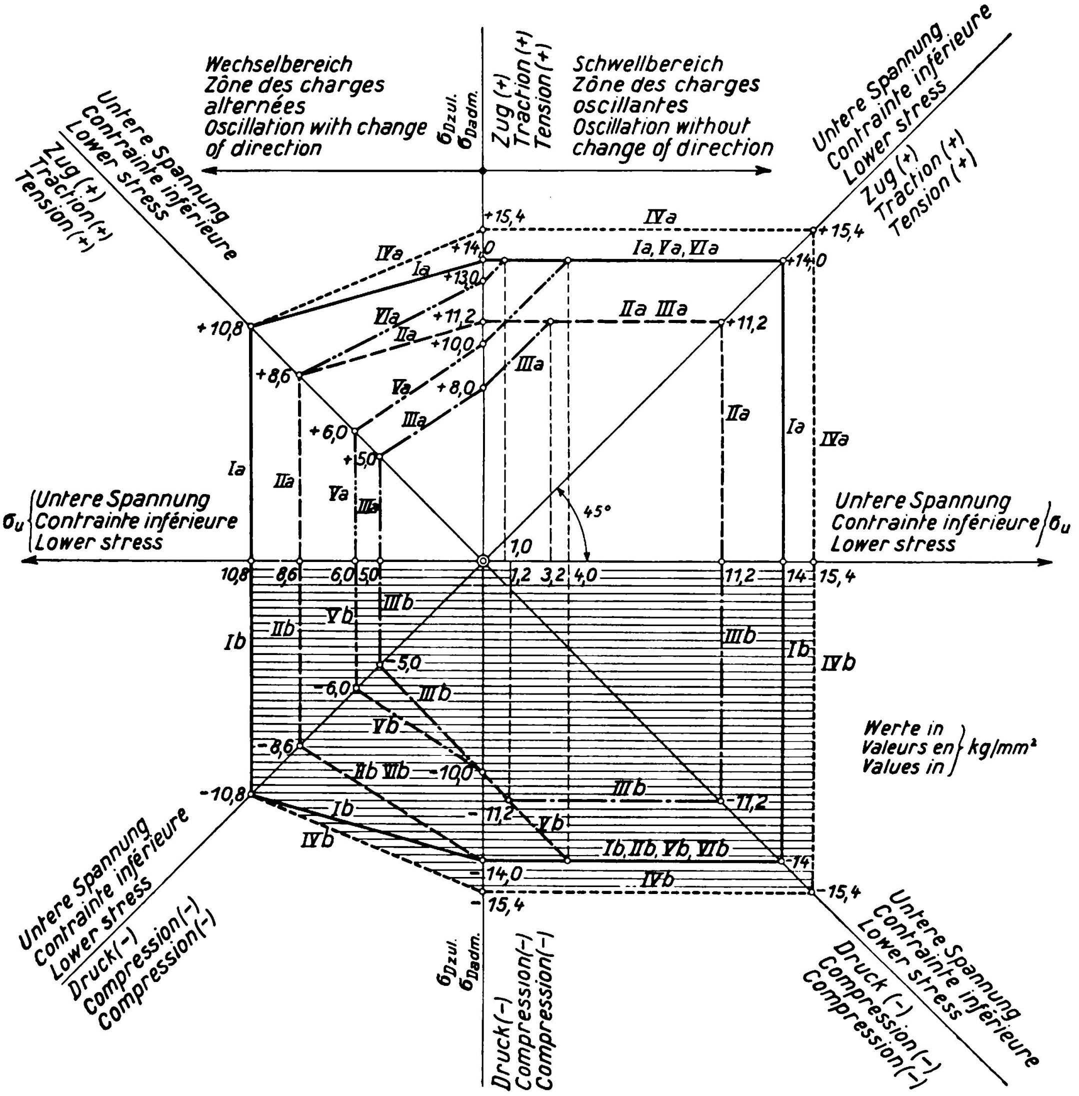


Fig. 1 V.

Diagram of permissible stresses $\sigma_{D_{zul}}$ 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.

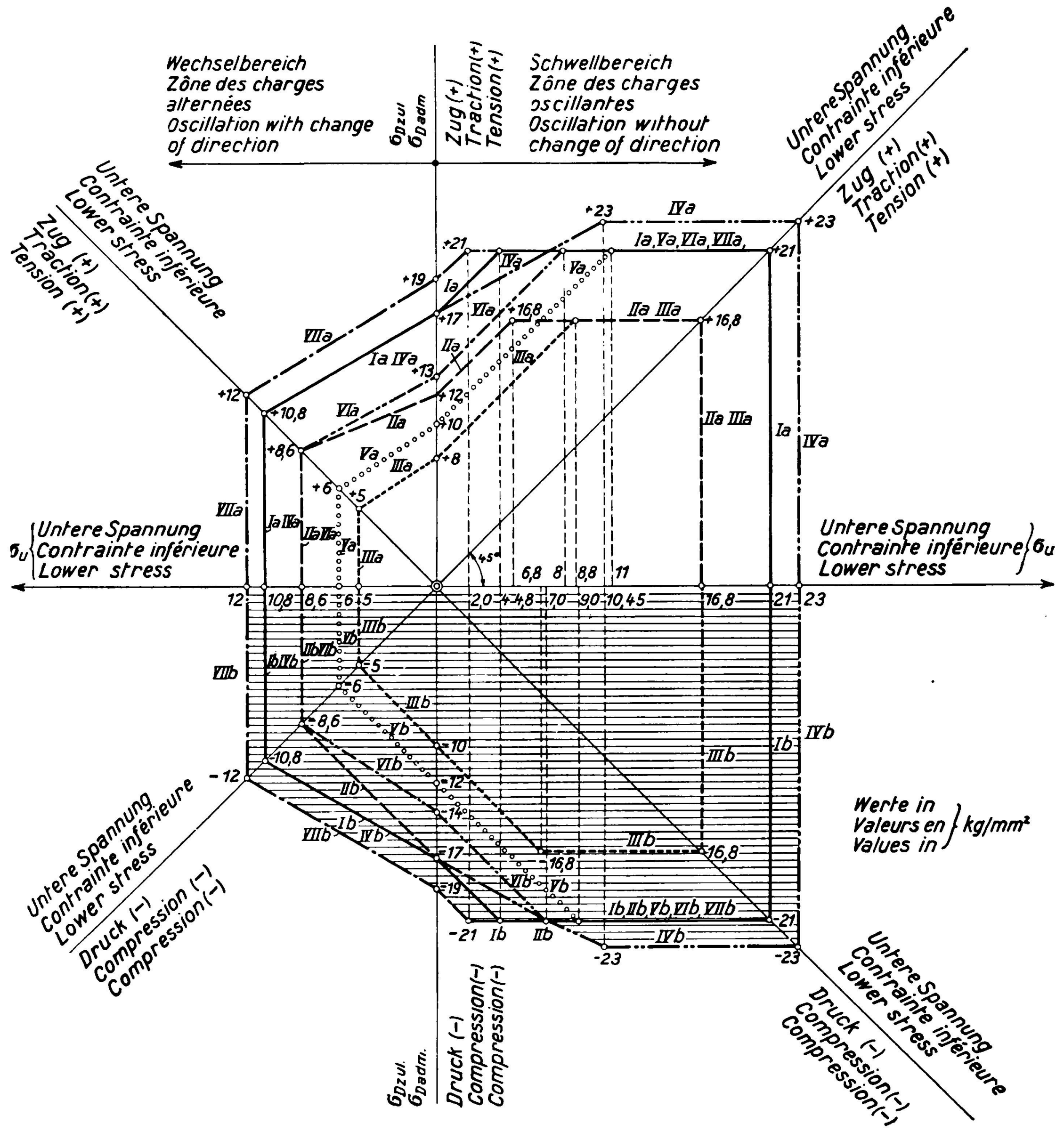


Fig. 2 V.

Diagram of permissible stresses $\sigma_{D\,zul}$ 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

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

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_{znl} 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 45⁰ with rewelded roots and gradual transition between weld metal and plate, should be arranged in tensile members, and hence we have

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

For the zone of compression the line IIb

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

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

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

In the tensile as well as in the compressive zone the lines $\sigma_{D\,zul}$ shall be under an angle of 45⁰, 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

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

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).

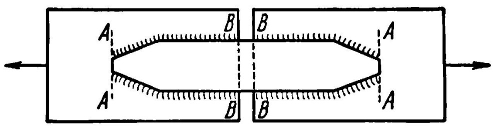


Fig. 30.

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 illustrations 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

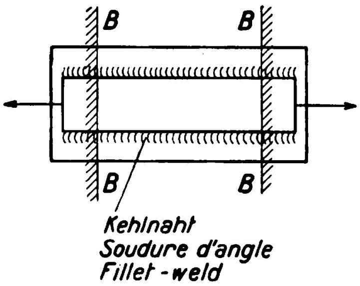


Fig. 31.

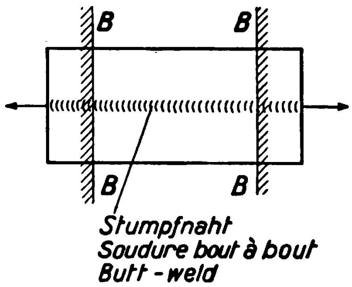


Fig. 32.

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

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

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

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 (BE, Para. 36) as for riveted bridges in which case the formula

$$\sigma' = \frac{\gamma \max M_I}{W_n} \text{ could apply.} \tag{4 V}$$

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

$$\sigma_1 = \frac{\gamma \max M_I}{\alpha W_n} \leqq \sigma_{zul} \tag{5 V}$$

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

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

2) *The coefficients* γ.

Based on fig. 33 we have

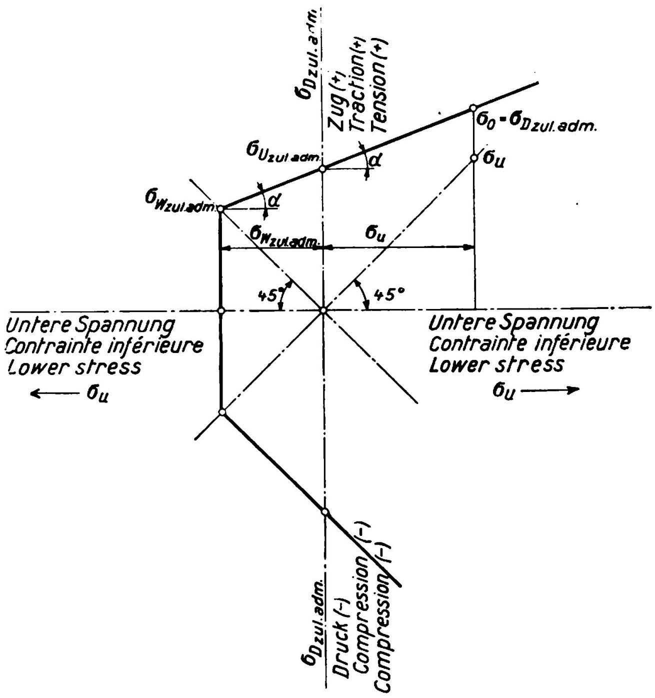


Fig. 33.

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

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

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

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

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

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 $\begin{Bmatrix} 1400 \text{ kg/cm}^2 \text{ for steel 37} \\ 2100 \text{ kg/cm}^2 \text{ for steel 52} \end{Bmatrix}$.
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 $\dfrac{\min M_I}{\max M_I}$. The values γ can be calculated with $\sigma_{U\,zul}$ and $\sigma_{W\,zul}$ for any conditions guiding the ratio $\dfrac{\min M_I}{\max M_I}$. The simplest way is as below for bridges in steel 37 we receive

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

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

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 $\dfrac{\min M_I}{\max M_I} = -1$, 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).

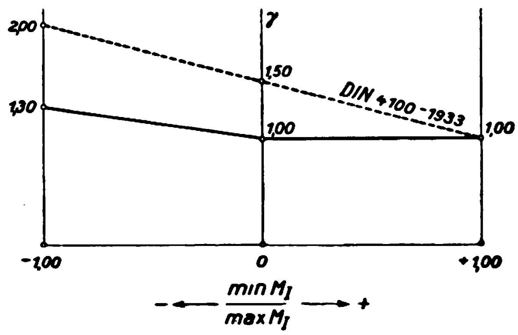


Fig. 34.

γ-lines for St. 37.

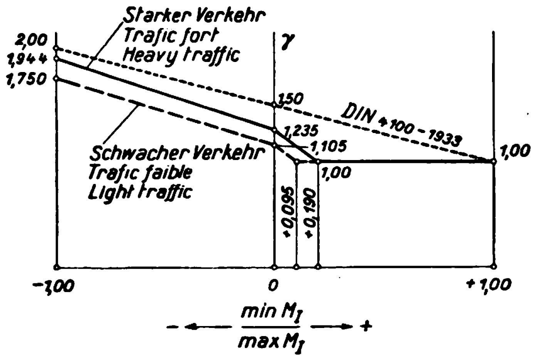


Fig. 35.

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

$$\sigma_{D\,zul} = \frac{\max M_I}{W_n} \tag{7 V}$$

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

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

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

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

(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 kg/mm² 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/cm^2 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	2	3	4	5		6
No.	Structural detail and type of weld	Kind of stressing	Range of alternating efforts	α values Range of surging efforts		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 $\tau_I' = \frac{\gamma \max Q_{Ix}}{\alpha\ t\ h_s} \leqq \sigma_{zul}$	0,65	0,65		
12	fillet-welds for girder connections stiff in bending	Main stress $\sigma = \frac{1}{\alpha} \sqrt{\sigma_I^2 + \tau_I^2} \leqq \sigma_{zul}$	0,75	0,75		
13		shear stress $\tau_I' = \frac{\gamma \max A_I}{\alpha\ \Sigma\ (al)} \leqq \sigma_{zul}$	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.)

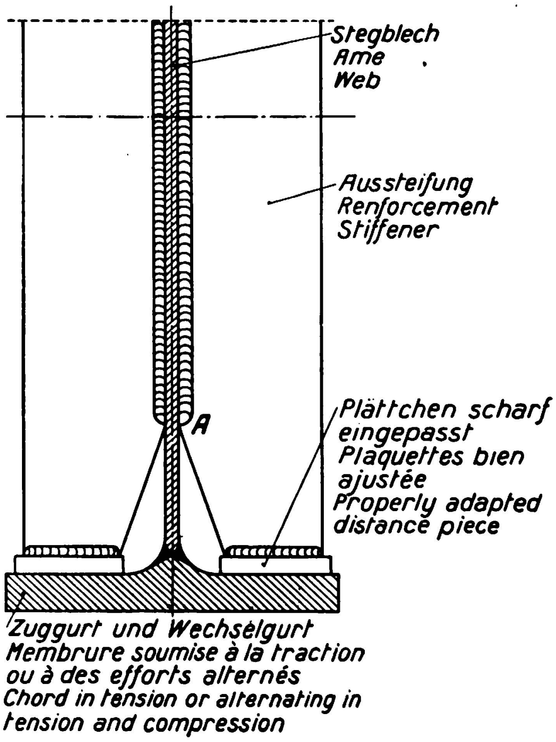

Fig. 12 V.

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.

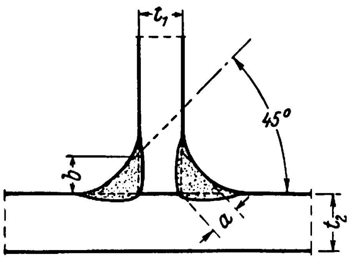

Fig. 13 V.

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

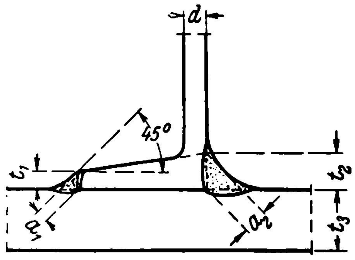

Fig. 14 V.

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

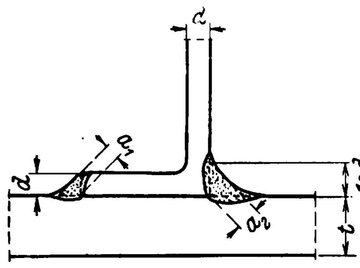

Fig. 15 V.

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

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.

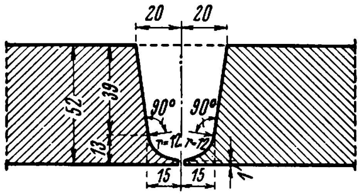

Fig. 36.
Rewelding, example of a U-shaped weld for a plate 52 mm thick.

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.

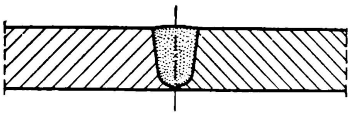

Fig. 16 a V.

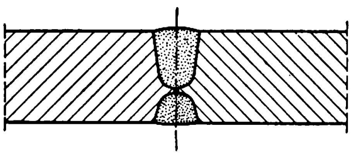

Fig. 16 b V.

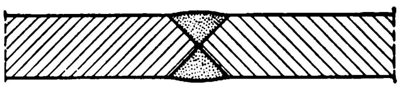

Fig. 17 V.

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.

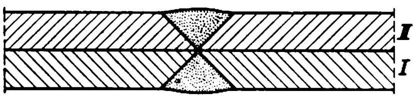

Fig. 18 V.
Two V-shaped welds.

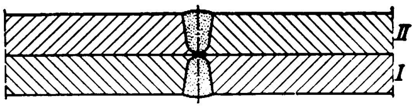

Fig. 19 V.
Two U-shaped welds.

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.

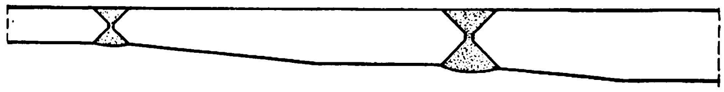

Fig. 20 V.

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.)

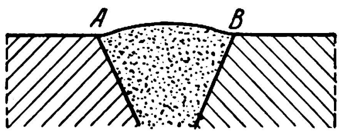

Fig. 25 V.

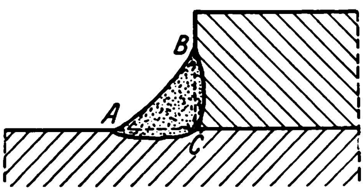

Fig. 26 V.

[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.

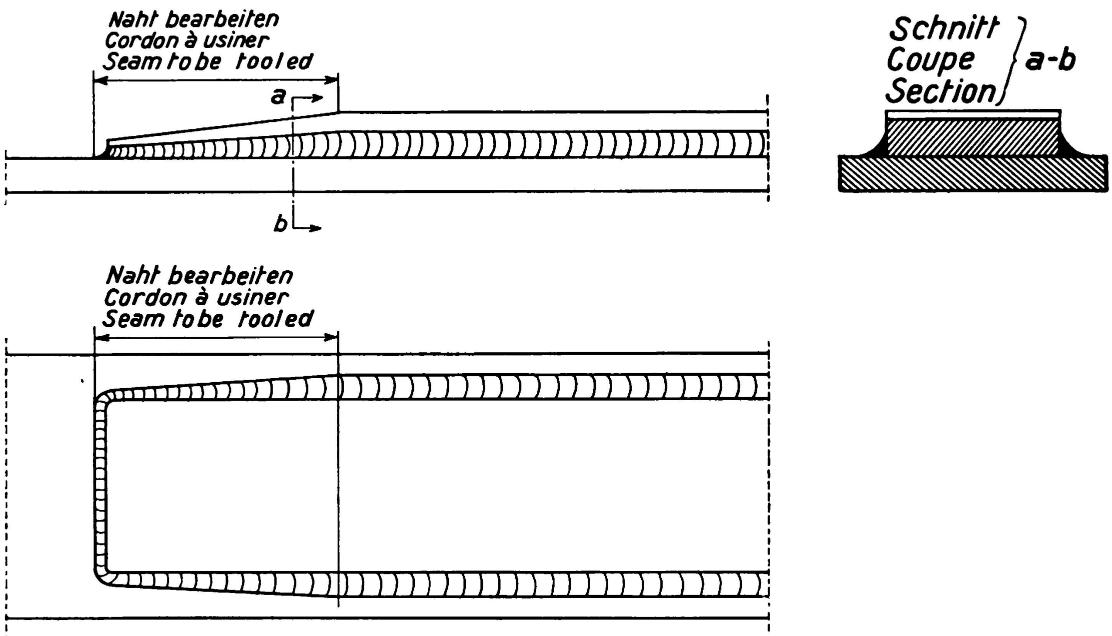

Fig. 27 V.

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.

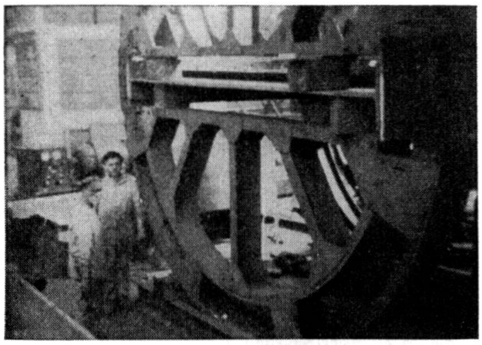

Fig. 37.

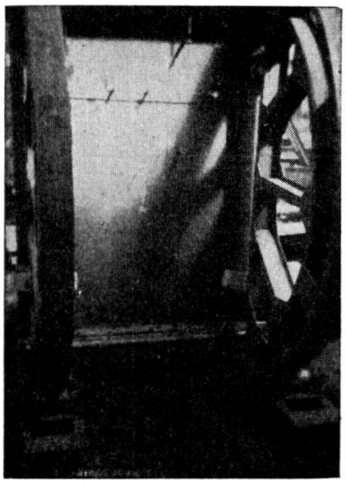

Fig. 38.

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.

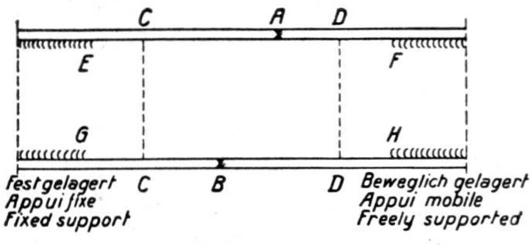

Fig. 39.

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 CC DD 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.