Fatigue strength and safety of welded structures (bridges, structural steel work and pressure pipes)

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IIIa 2

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

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

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

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I. Introduction.

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

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

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

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

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

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

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

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

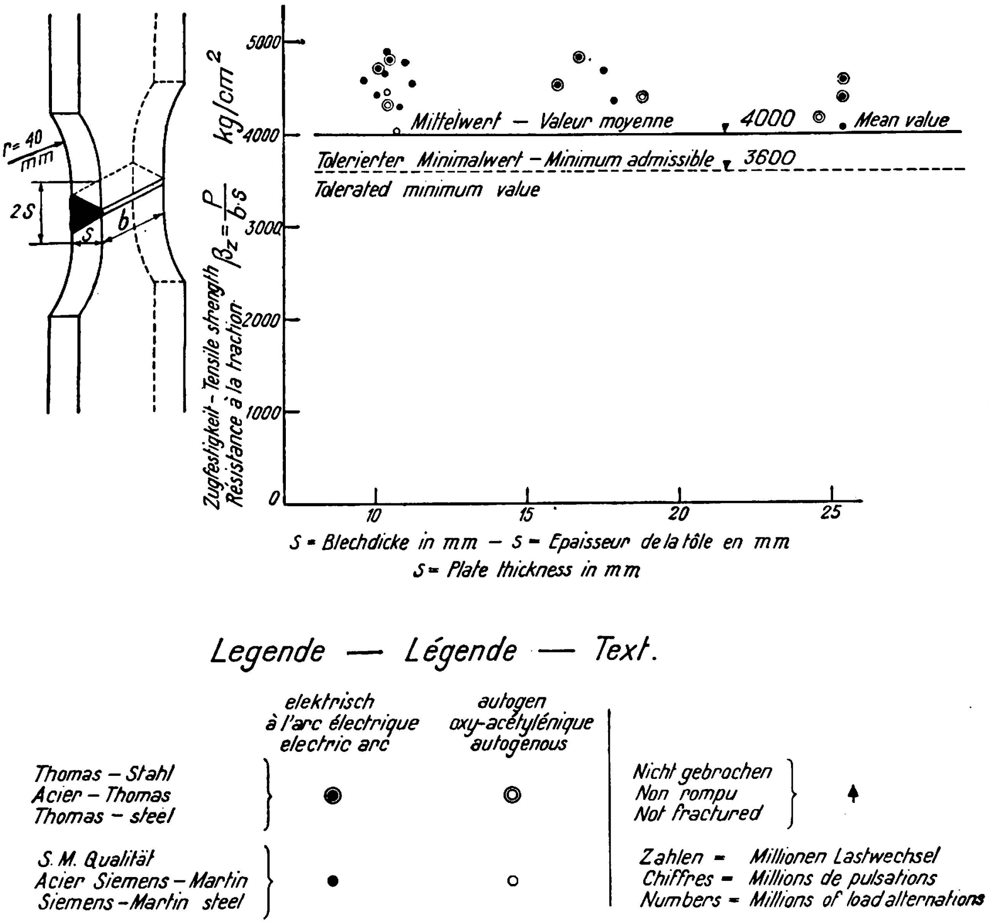

Fig. 1.

Butt weld — Tensile strength.

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

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

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

Fatigue Strength:

$$\sigma_D = \text{Oscillation strength} \begin{cases} \sigma\max = +\sigma_D \\ \sigma\min = -\sigma_D \end{cases}$$

$$\sigma_U = \text{Surge load strength} \begin{cases} \sigma\max = +\sigma_U \\ \sigma\min = 0 \end{cases}$$

$$\frac{1}{2}\sigma_W = \text{Alternating strength} \begin{cases} \sigma\max = +\sigma_W \\ \sigma\min = +\frac{1}{2}\sigma_W \end{cases}$$

visible disturbance of molecular equilibrium . . . . . σ_f = limit of flow

maximum tensile strength, in its relation to the original cross-sectional area . . . . β_z = tensile strength

Table 1.

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

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

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

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

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

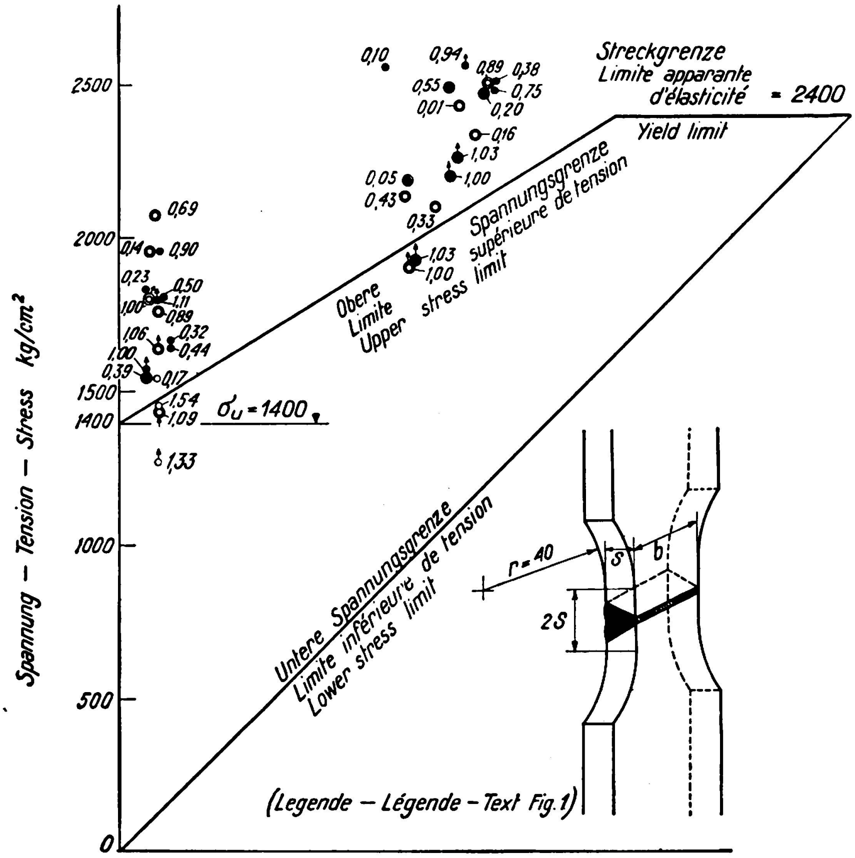

Fig. 2.
Butt weld — Tension-fatigue test.

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

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

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

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

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

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

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

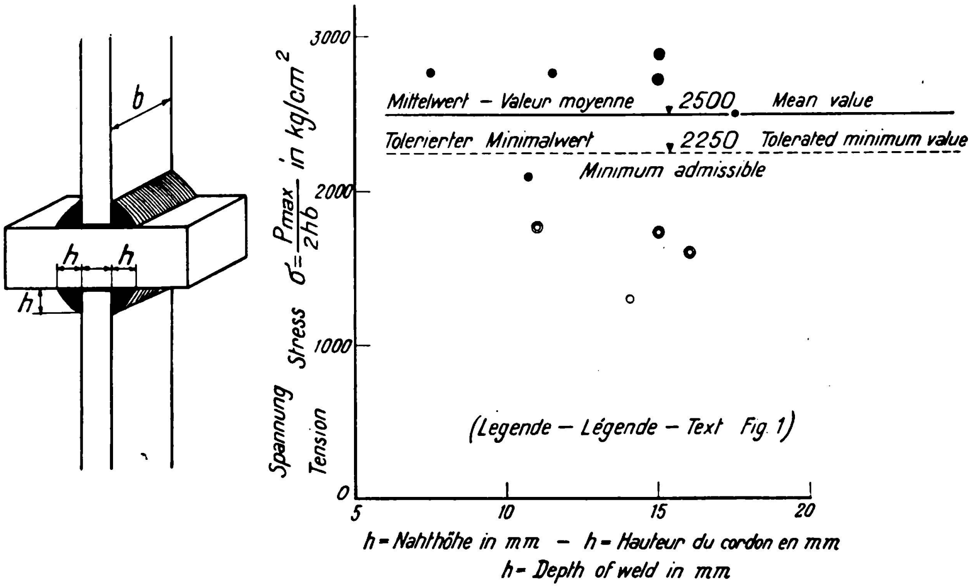

Fig. 3.

Fillet weld — Tensile strength.

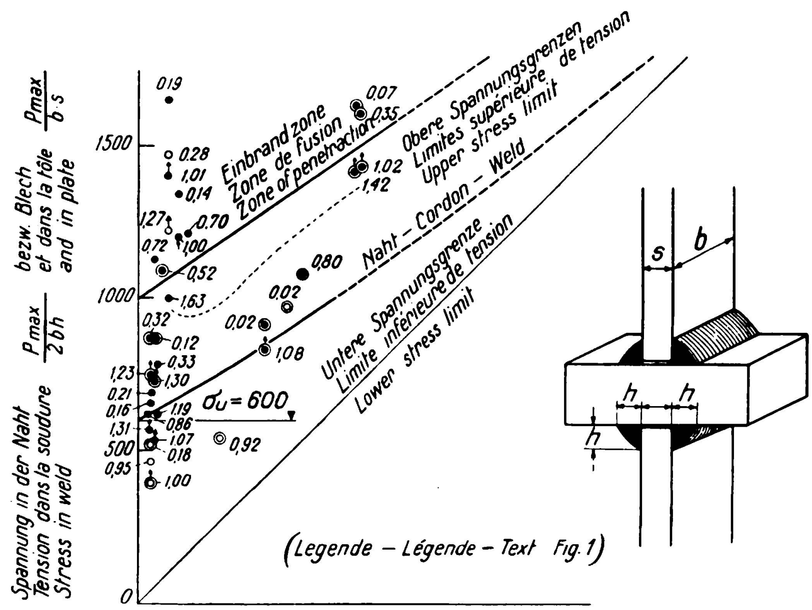

Fig. 4.

Fillet weld — Tension-fatigue test.

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

Table 2.

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

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

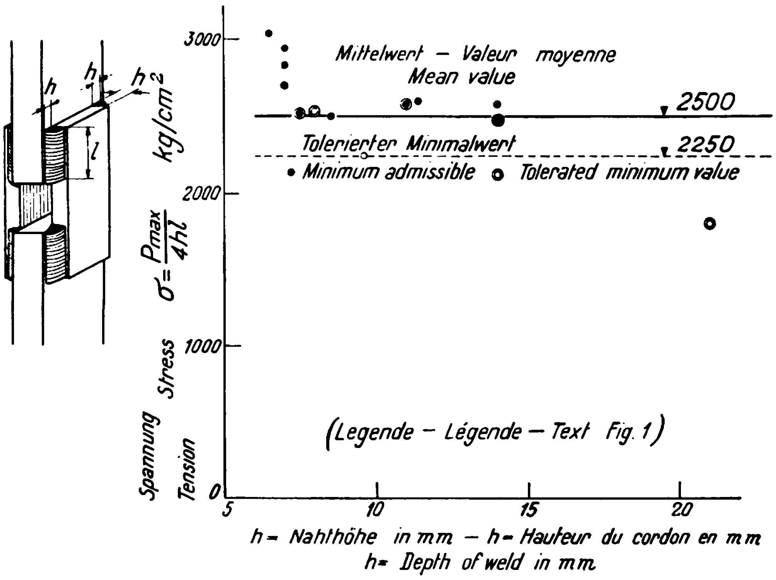

Fig. 5.

Side fillet weld — Tensile Strength.

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

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

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

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

III. Permissible stresses.

For welded connections of standard structural steel ($\beta_z \cong 4000$ kg/cm^2) the following stresses are permissible (kg/cm^2):

Table 3.

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

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

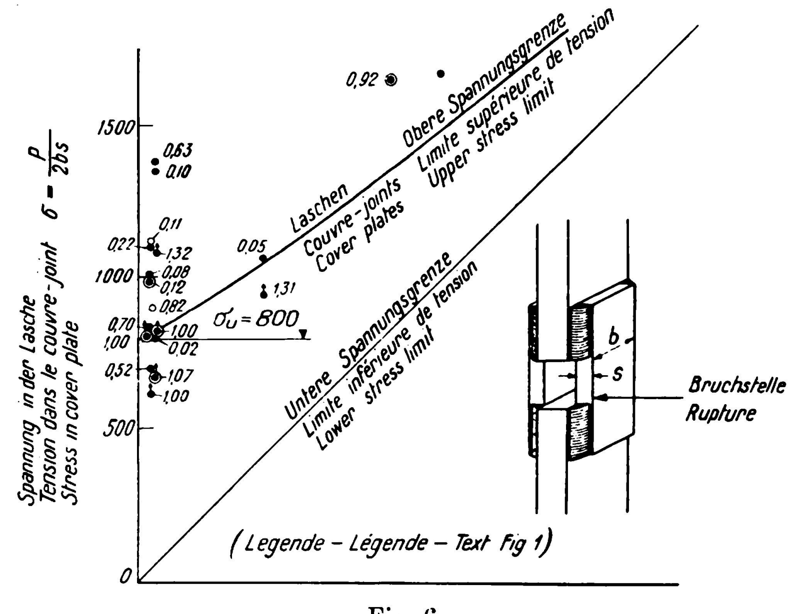

Fig. 6.

Side fillet weld — Tension-fatigue test.

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

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

Table 4.

Ratios[5]

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

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

---

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

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

Butt-weld

	Tension	Compression	Shear
unannealed	$\alpha_1 = \frac{1400}{1900} \cong 0{,}7$	$\alpha_1 = 1{,}0$	$\alpha = \frac{1100}{1900} \cong 0{,}55$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	
annealed	$\alpha_1 = \frac{1500}{1900} \cong 0{,}8$		
	$\alpha_2 = \frac{1800}{1900} \cong 0{,}95$		— Fig. 13 —

Fillet, side and end fillet welds.

	Tension	Compression	Shear
	$\alpha_1 = \frac{700}{1900} \cong 0{,}35$	$\alpha_1 = 0{,}5$	$\alpha \cong \frac{750}{1900} = 0{,}4$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	— Fig. 14 —

Zone of penetration.

	Tension	Compression	Shear
Butt-welds	$\alpha_1 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_1 = 1{,}0$	$\alpha \cong 0{,}58$
	$\alpha_2 = \frac{1610}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	
Fillet welds (group)	$\alpha_1 = \frac{1100}{1900} \cong 0{,}6$	$\alpha_1 = 0{,}9$	$\alpha \cong 0{,}53$
	$\alpha_2 = \frac{1600}{1900} \cong 0{,}85$	$\alpha_2 = 1{,}0$	— Fig, 14 and 15 —

For pure *compression* in butt-welds, without danger of buckling, the fatigue strength is considerably higher than for pure tension: the yield limit due to the thermal influence is only slightly lower ($\sigma_f = 2400$ kg/cm^2 as compared to

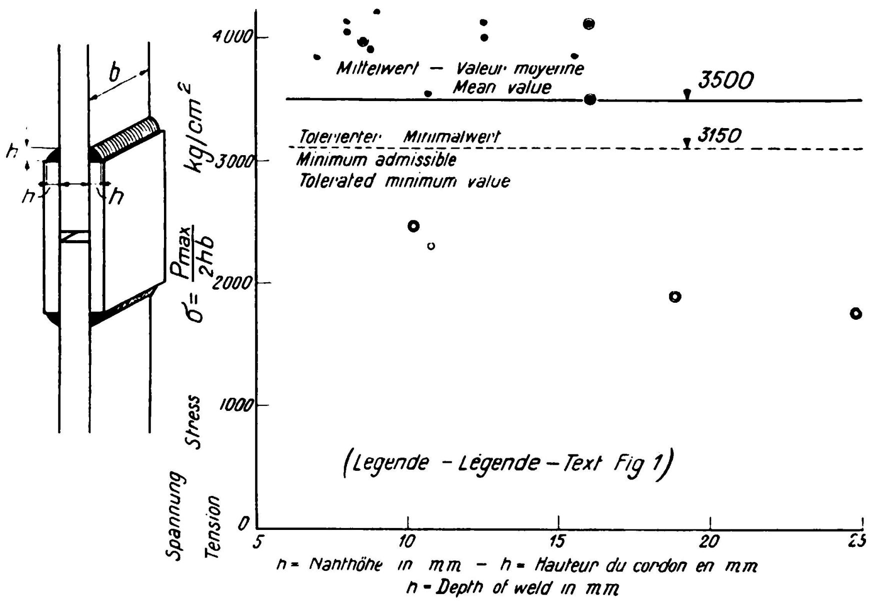

Fig. 7.

Front fillet weld — Tensile strength.

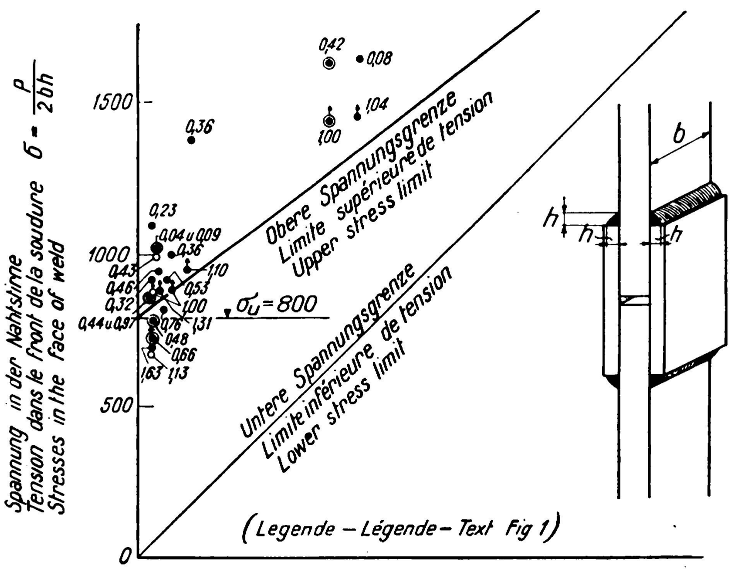

Fig. 8

Front fillet weld — Tension fatigue test.

$\sigma_f = 2600$ kg/cm^2) so that it is quite in order to admit a permissible compression stress for riveted connections of standard structural steel of $\beta_z = 4000$ kg/cm^2.

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

Table 5.

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

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

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

which is in concordance.

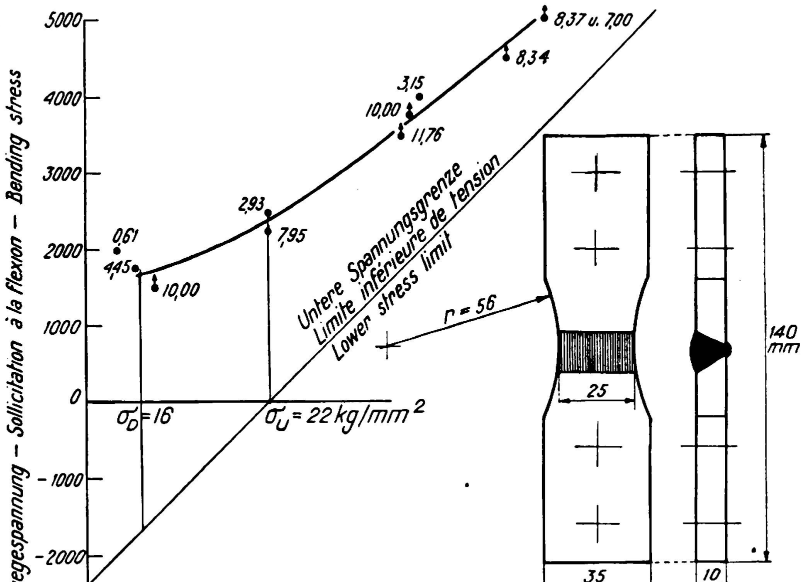

Fig. 9.

Butt weld with bulging surface — Fatigue bending test.

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

Table 6.

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

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

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

higher than that allowed for tension.

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

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

Table 7.

Ratios[5] A = 0

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

IV. Safety factor.

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

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

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

which is based on the constance of deformation energy

Table 8.

Bridge Construction.

Butt-welds in tension

(Figs. 16 and 17)

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

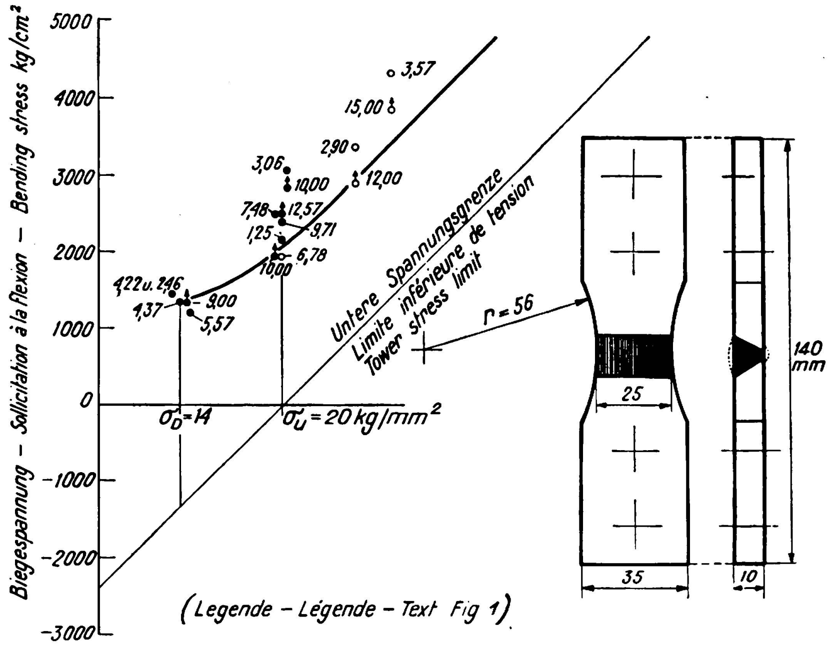

Fig. 10.

Butt weld with protrusion machined off — Fatigue bending test.

Structural Engineering.

Butt-welds in tension

(Figs. 16 and 17)

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

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

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

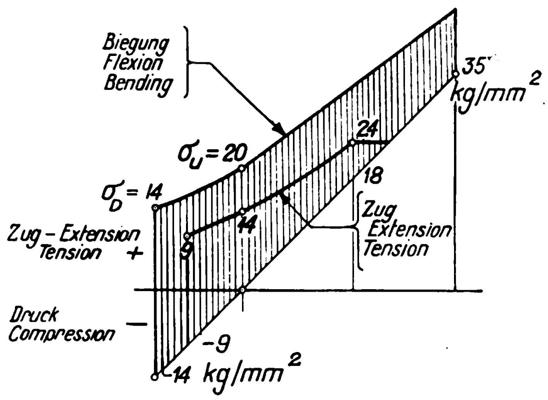

Fig. 11.
Butt welds
Fatigue strength

Table 9.

Bridge Construction.

(Fig. 15)

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

Structural Engineering[1].

(Fig. 15)

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

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

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

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

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

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

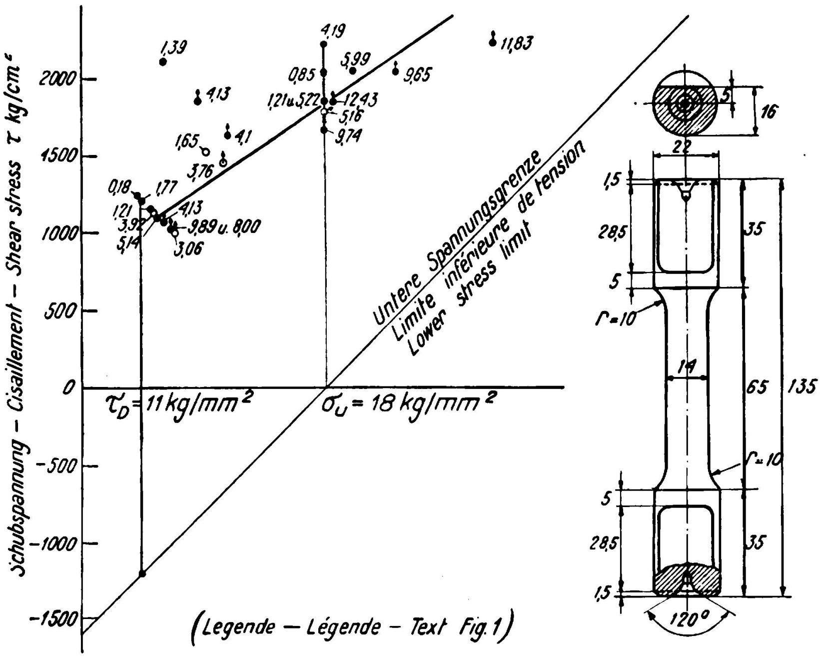

Fig. 12.

Torsion — Fatigue Tests.

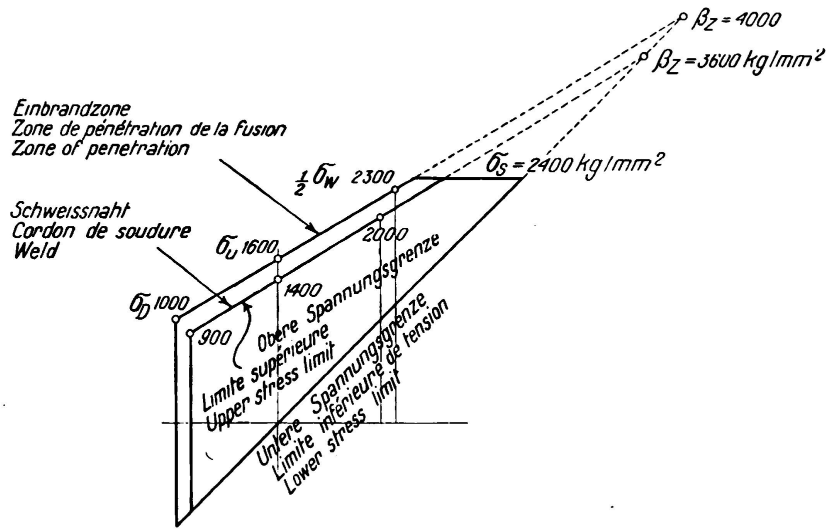

Fig. 13.

Butt welds — Limits of fatigue.

V. Methods of calculation.

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

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

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

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

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

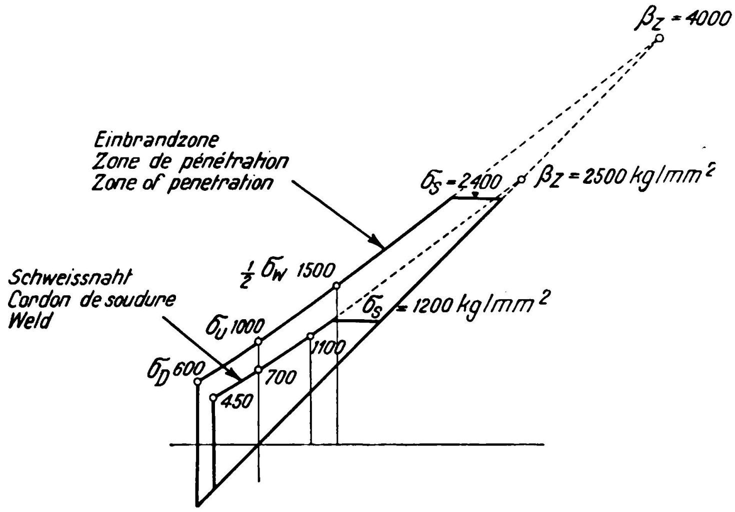

Fig. 14.

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

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

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

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

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

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

Butt-welds.

Case 1.

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

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

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

of which the higher value is decisive.

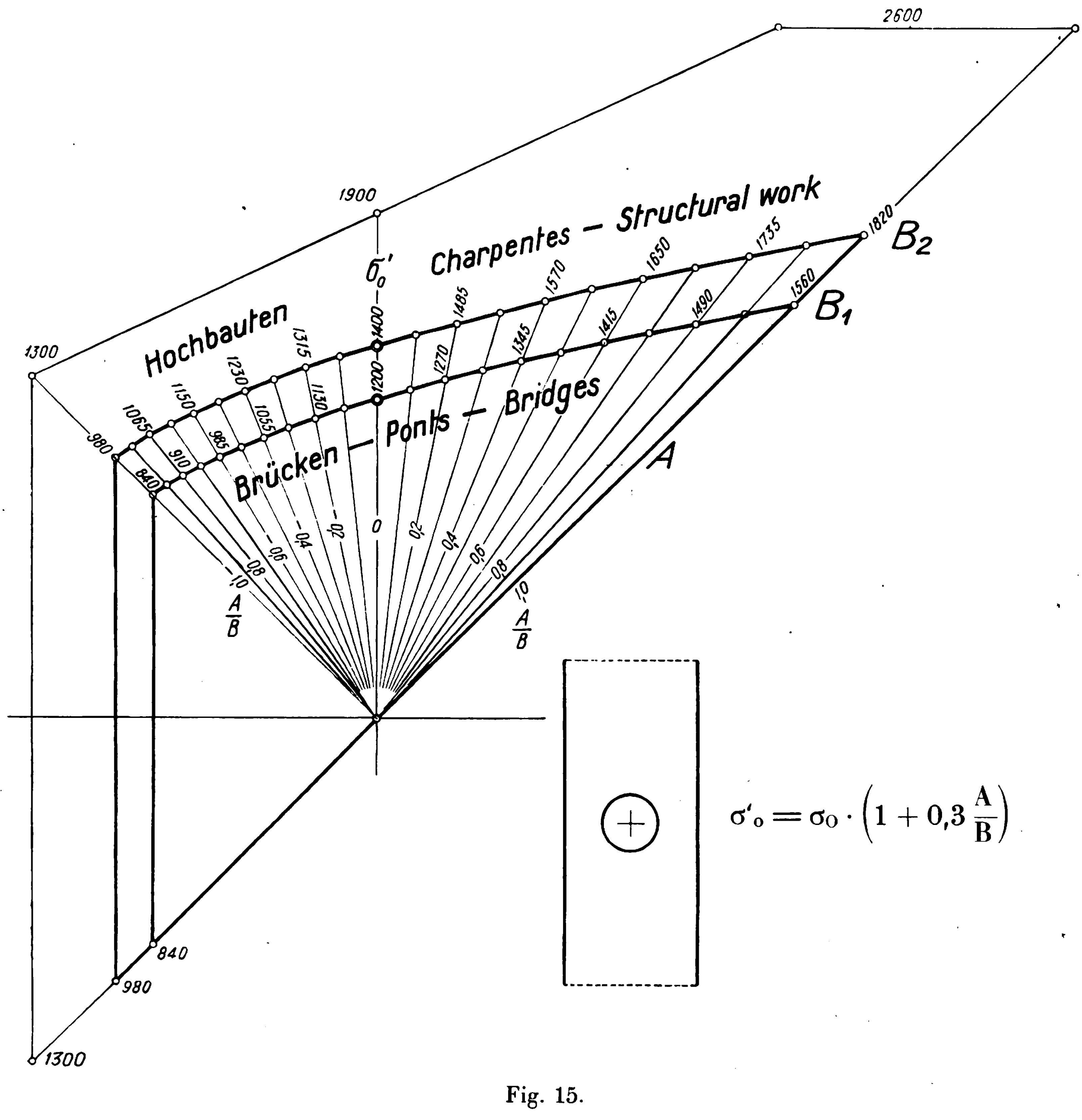

Fig. 15.

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

Case 2.

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

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

(Fig. 19)

The figures for stressing due to surge load strength are

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

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

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

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

$$\sigma_{o\,zul} = 1200 \text{ kg/cm}^2 \text{ for bridges, or}$$

$$\sigma_{o\,zul} = 1400 \text{ kg/cm}^2 \text{ for other structural steel work}^{10}.$$

(zul = permissible.)

See Table 9.

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

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

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

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

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

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

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

Fillet welds.

Case 3.

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

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

and

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

(Fig. 20)

the higher of the two values for σ_g is decisive.

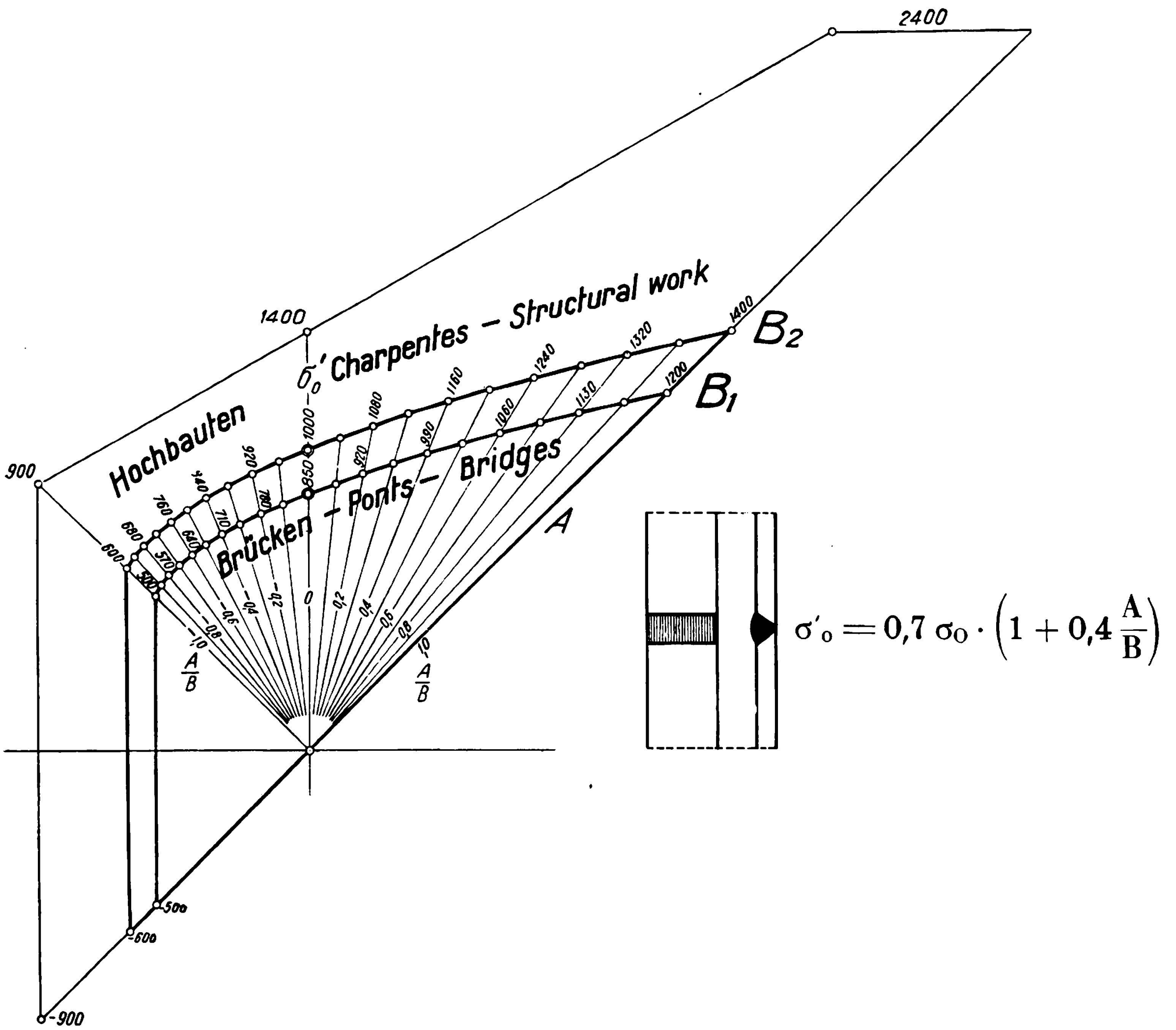

Fig. 16.

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

Case 4.

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

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

(Fig. 20)

The following are the figures for tension and compression:

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

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

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

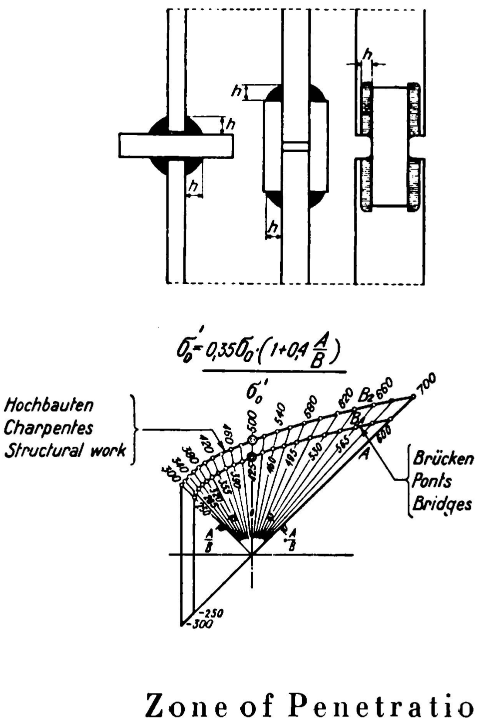

Fig. 17.

Fillet, side fillet and butt welds — Permissible stresses.

A = minimum
B = maximum.

Zone of Penetration.

Cases 5 and 6. — Butt-weld.

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

Case 7. — Fillet-weld.

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

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

and

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

(Fig. 21)

Case 8. — Fillet-weld.

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

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

(Fig. 21)

The following are the figures for tension and compression:

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

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

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

The most favourable position of the joint from the practical point of view is that under 45⁰ to the direction of the forces, in which case:

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

and hence according to equation (5)

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

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

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

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

for the joint under . . .	30^0	45^0	60^0
Amount of gain:	8 %	28 %	23 %

to the line at right angle to the direction of forces

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

The most favourable placing of the weld joint from a practical point of view is done under 60 ^{0} to the perpendicular line to the direction of forces P per unit length. Accordingly we receive

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

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

h = depth of weld.

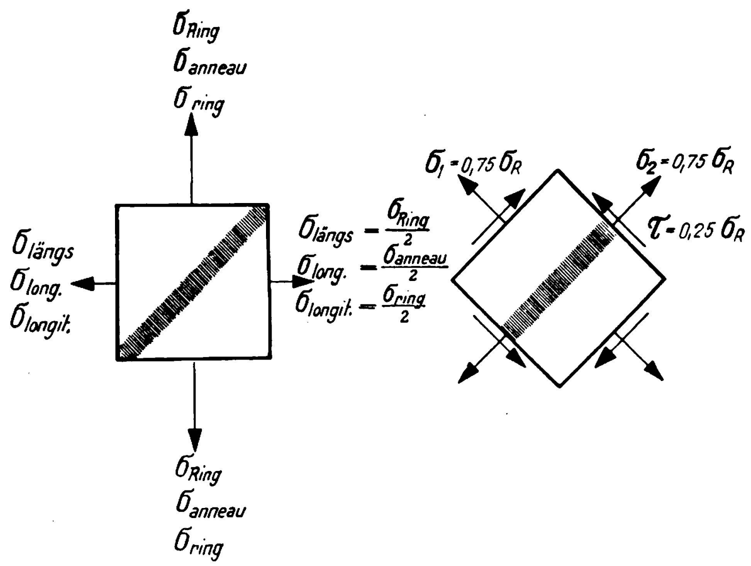

Fig. 18.
Helical weld — System of stresses.

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

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

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

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

Helical welds of containers, boilers and pressure pipes.

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

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

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

hence we receive

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

(Ring = annular, ring [ring stress])

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

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

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

VI. Experience.

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

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

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

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

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

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

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

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

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

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

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

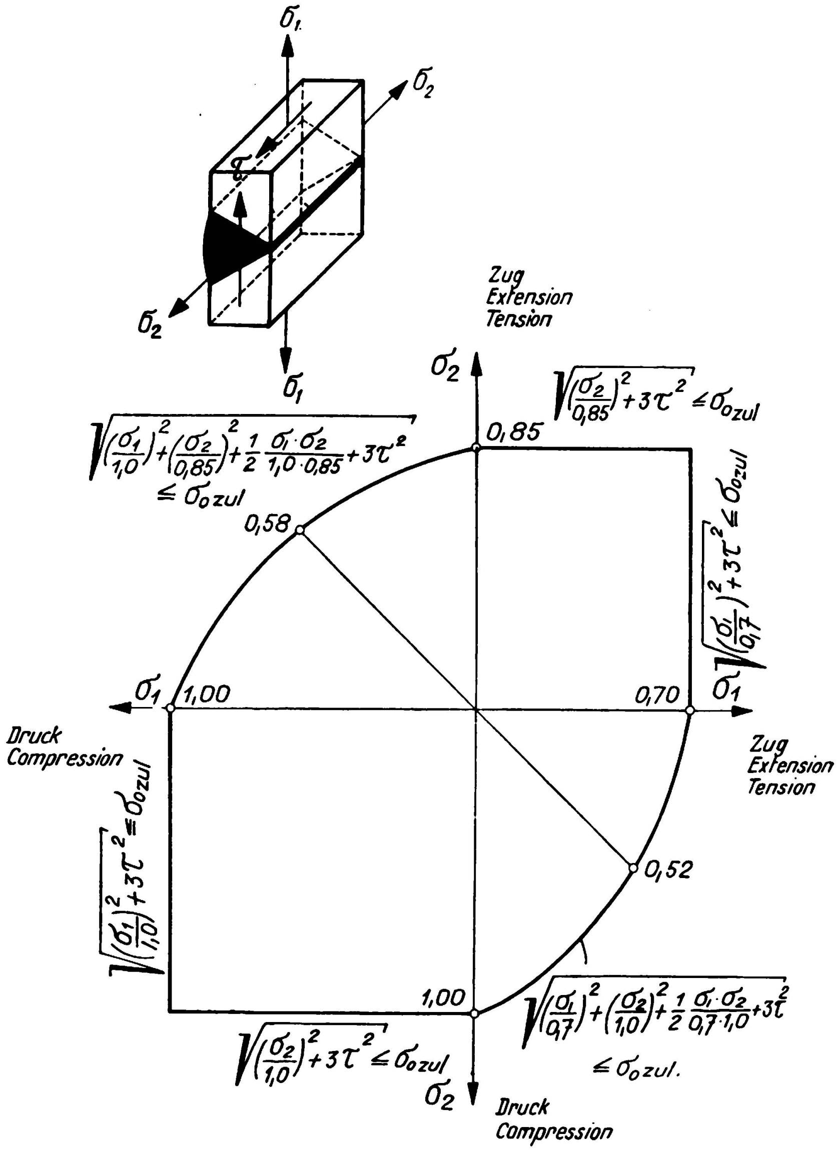

Fig. 19.

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

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

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

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

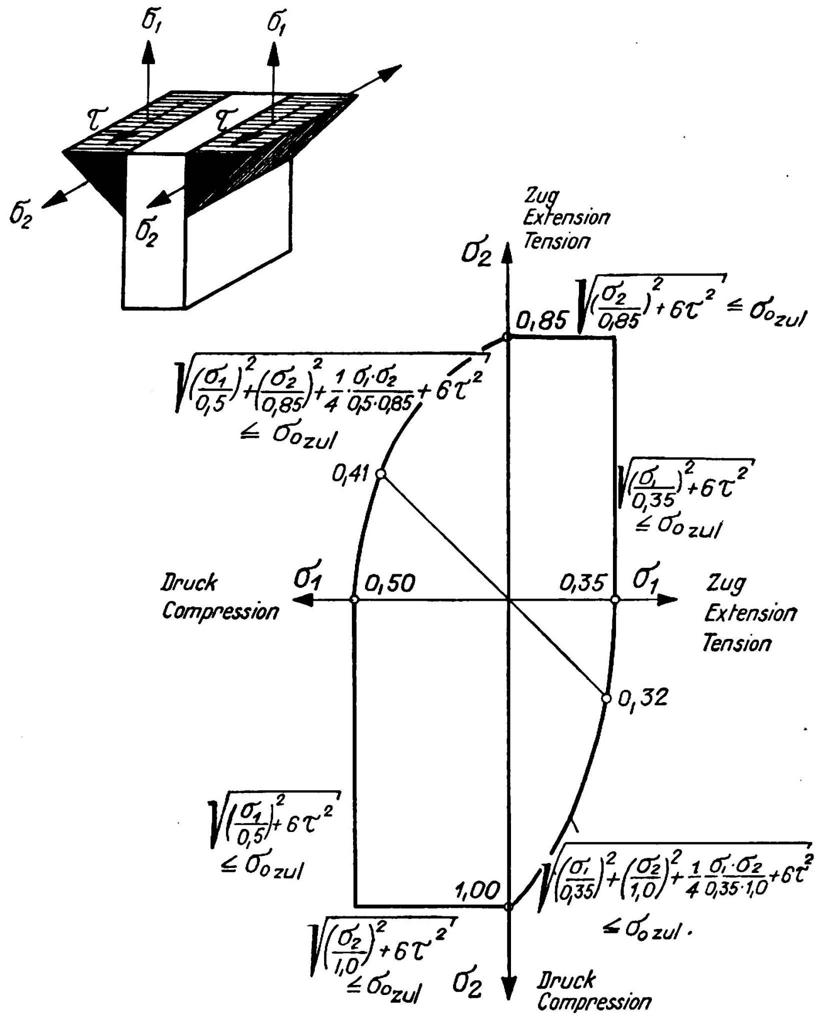

Fig. 20.

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

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

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

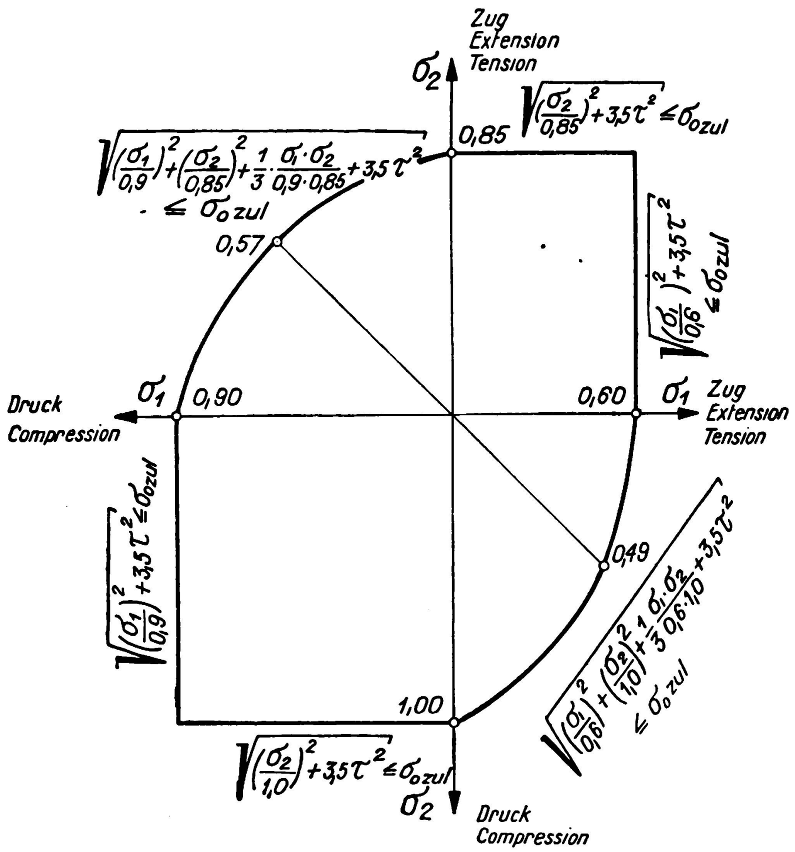

Fig. 21.

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

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

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

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

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

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

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

concentration of stresses and excessive and real fatigue stresses.

Summary.

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

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

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

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

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