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Stress distribution in fillet welds

Spannungsverteilung in Kehlnähten

Distribuição das tensões nas soldaduras de canto

Distribution des contraintes dans les soudures d'angle

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1. *Introduction.*

In steel trusses and similar framed structures, the bars or other members which meet at a joint are often attached to the connection plates (gusset plates) by means of fillet welds. In such joints, see e. g. Fig. 1, the forces are wholly or partly transmitted from the bar to the connection plate through shearing stresses τ. The practice is generally to design welds of this kind in such a manner as to take account of the shearing stresses alone. However, if the member welded to the connection plate is submitted to tension or compression, then tensile or compressive stresses $\sigma_{\parallel}$ which are parallel to the longitudinal direction of the weld must also occur in the weld. Furthermore, that tensile or compressive force in the bar which is transmitted through the fillet weld is eccentric with reference to the weld. Accordingly, the weld is also subjected to stresses $\sigma_{\perp}$ at right angles to the longitudinal direction of the weld. Therefore, the weld is in a multiaxial state of stress, and it is of importance to know whether the load-bearing capacity of the weld is also influenced by the stress components $\sigma_{\parallel}$ and $\sigma_{\perp}$.

In the Swedish standard specifications for welding of steel structures, just as in the analogous specifications in many other countries, the load-bearing capacity of welds is determined by the hypothesis of the maximum strain energy of distortion. This hypothesis can serve as a basis for calculating the comparison stress

$$\sigma_r = \sqrt{\left(\frac{\sigma_{\perp}}{\alpha}\right)^2 + \sigma_{\parallel}^2 - \frac{\sigma_{\perp}}{\alpha} \cdot \sigma_{\parallel} + 3\,\tau^2}$$

which shall not exceed the allowable stress in the parent metal. In this formula, α is a form factor which is less than unity. The standard specifications do not state that the formula in question shall be applied to the case illustrated in Fig 1. If it is nevertheless desired to find out whether the formula is applicable to this case, then it is necessary to know how the stresses set up in the weld are distributed in its longitudinal direction.

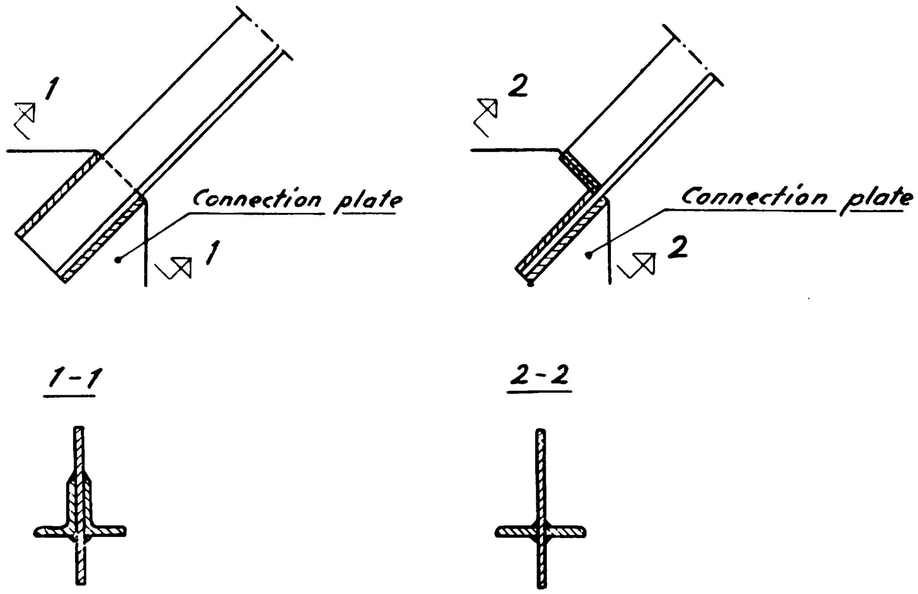

FIG. 1

In order to clarify these questions to a certain extent, two series of tests on fillet weld joints of the above-mentioned type were made at the Royal Institute of Technology, Division of Structural Engineering and Bridge Building, Stockholm, Sweden. The purpose of the tests contained in the first series was to determine the variation in the ultimate load with the design of the members attached to a connection plate, so as to give an idea of the effects produced by the stresses $\sigma_{\parallel}$ and $\sigma_{\perp}$. The tests comprised in the second series were made in order to determine the shearing stress distribution in a fillet weld.

2. *Variation in Ultimate Load with Design of Members Attached to Connection Plates.*

The test specimens used in this test series are shown in Fig. 2. The test specimens were submitted to tensile forces. In order that the test conditions should be as simple and well-defined as possible, the members welded to the connection plates were made of bars of rectangular cross section, while the «connection plates» were uniform in width, and their cross-sectional area was equal to the total cross-sectional area of the bars. The test specimens comprised bars of five different cross-sectional shapes, see Fig. 2. In the test specimens a, b and c, the bars were equal in cross-sectional area, but differed in height H and in width B.

These test specimens were employed with a view to throwing light on the effect of the stress $\sigma_{\perp}$ acting at right angles to the longitudinal direction of the weld. It was expected that the stress $\sigma_{\perp}$ would reach a maximum value in the test specimen a and a minimum value in the test specimen c. In the tests specimens c, d and e, the cross-sectional area of the bars was varied, whereas the height H was constant. These test specimens were used in order to obtain information on the effects of the tensile stresses $\sigma_{\parallel}$ acting in parallel to the longitudinal direction of the weld. It was presumed that $\sigma_{\parallel}$ would reach a maximum value in the test specimen e and a minimum value in the test specimen d. Six specimens of each of these five types were prepared and subjected to the tests. The steel

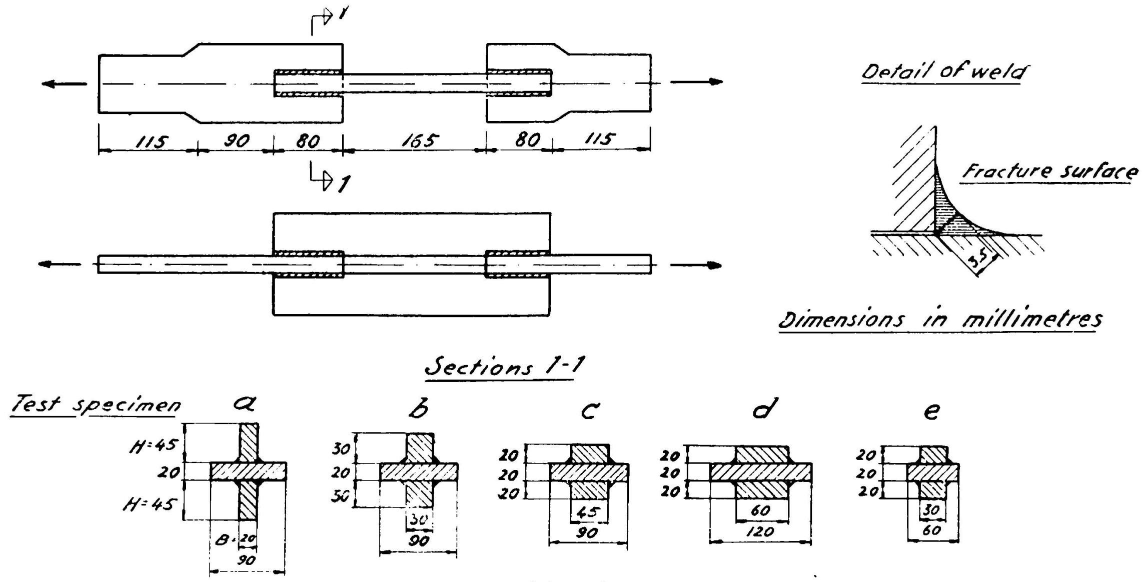

FIG. 2

was supplied in seven bars. The yield point stress and the ultimate strength were determined for each of these bars. The mean value of the yield point stress was found to be 2820 kg per cm^2 (the observed values varied from 2780 to 2890 kg per cm^2). The mean value of the ultimate strength was found to be 5390 kg per cm^2 (the observed values varied from 5230 to 5470 kg per cm^2).

It was intended to test only one of the two weld joints on each test specimen, and the strength of the other weld joint was increased so as to prevent failure. The joints to be tested were as far as possible welded in a similar manner on all 30 test specimens. All four welds were 80 mm in length, and the dimension a was equal to 3.5 mm, see Fig. 2. The maximum deviation was 3 mm in weld length and $\pm$ 0.2 mm (estimated value) in dimension a. In order that the dimension a should be as accurate as possible, the whole weld was ground. The welds were made by means of Philips Ph 48 electrodes. The yield point limit and the ultimate strength of the weld metal were determined on a small test bar, 2.7 mm by 2.7 mm in cross section, cut out of one of those welds which were not intended for testing. The yield point stress was found to be 4700 kg per cm^2, the ultimate strength was 5750 kg per cm^2, and the ultimate elongation in a gauge length of 65 mm was 11 per cent.

The specimens were tested in a tension testing machine having a capacity of 50 tons. The ultimate loads observed in these tests are given in Table 1. For two specimens, b1 and c5, the load of 50 tons proved insufficient to cause failure. All other specimens failed either in two or in four welds. The failure was in shear along the surface of minimum thickness of the weld, see Fig. 2. A similar type of failure was observed in the second series of tests. A test specimen after failure in the second test series is shown in Fig. 5.

TABLE 1

Observed Ultimate Loads

Test Specimen		Ultimate Load			Mean Ultimate Shearing Stress kg per cm²
Type	No.	Per Test Tons	Mean Value per Type Tons	Average Deviation Tons	
a	1 2 3 4 5 6	49.2 43.3 38.3 48.0 47.0 41.4	44.5	4	3940
b	1 2 3 4 5 6	> 50.0 44.3 43.5 40.6 46.4 44.3	> 44.8	3	> 4000
c	1 2 3 4 5 6	48.2 42.0 44.7 42.2 > 50.0 47.3	> 45.8	3	> 4100
d	1 2 3 4 5 6	41.1 41.2 44.2 45.3 40.0 49.4	43.5	3	3900
e	1 2 3 4 5 6	41.7 40.3 42.8 48.2 39.5 40.9	42.2	3	3780

The dispersion of the ultimate loads observed on test specimens of the same type is relatively large. The greatest deviation from the mean value for an individual test specimen is 14 per cent. On the other hand, the difference between the mean values relating to different types of test specimens is comparatively small. The greatest difference is 8 per cent. It is therefore impossible to use these tests as a basis for drawing any conclusions as how the design of the bars attached to the connection plates influences the load- bearing capacity of the welds. In this case, however, it seems that the variation in the design of the bars, and hence in the stresses $\sigma_{\parallel}$ and $\sigma_{\perp}$, produced only a slight effect on the load-bearing capacity of the welds. It is to be supposed that this capacity was indirectly determined by the shearing stresses set up in the welds. It appears to be obvious that the stresses $\sigma_{\parallel}$ actually occur, and the fact that the stresses $\sigma_{\perp}$ vary with the type of test specimen was established by measuring the stresses (or rather the strains) at some points close to the weld joints on several test specimens. When these measurements were made on test specimens of the type a, it was found that the resultant of the tensile stresses in one of the bars, at the front edge of the weld, had an eccentricity of 0 to 4 mm (reckoned in the direction towards the welds) with reference to the centre line of the bar at half-height. This implies that the eccentricity of the point of application of the tensile force with respect to the weld was about 20 mm. In the test specimens of the type c, which were provided with bars 20 mm in height, the eccentricity of the tensile force must have been materially smaller.

The mean shearing stress at failure in the tests was about 4000 kg per cm^2. The corresponding comparison stress calculated from the above-given formula without taking $\sigma_{\parallel}$ and $\sigma_{\perp}$ into account, i. e. $\sigma_r = \sqrt{3} \cdot \tau$, is about 7000 kg per cm^2. The maximum values of the shearing stress and the comparison stress may be presumed to be substantially higher. Consequently, it seems that the load-bearing capacity in this case is not inconsiderably greater than that value which would result from a calculation of the comparison stress, irrespective of whether this stress is computed from the actual stresses or from a mean value of some kind or other.

In this connection it is to be noted, however, that the expression for the comparison stress (in which σ_r is put equal to the yield point stress) states only a condition for yield, and not a condition for failure. It is quite possible that yield occurred in the welds before failure, but no large displacements of the bars with reference to the «connection plate» were observed in the tests. The order of magnitude of the greatest actual displacements was 0.1 mm.

3. *Shearing Stress Distribution along Welds.*

The main purpose of the second test series was to give an idea of the variation in the shearing stress distribution along a fillet weld with the length of weld and the thickness of weld. The test specimens used in this series, see Fig. 3, were in principle similar in design to those of the type a in the first series, but the gripping devices were modified.

The test specimens were six in number. The welds were varied in length and in thickness, see Table 2.

Just as in the first test series, only one of the weld joints was intended for testing. All four welds of this joint were made so as to be as similar as possible, and were ground to required dimensions.

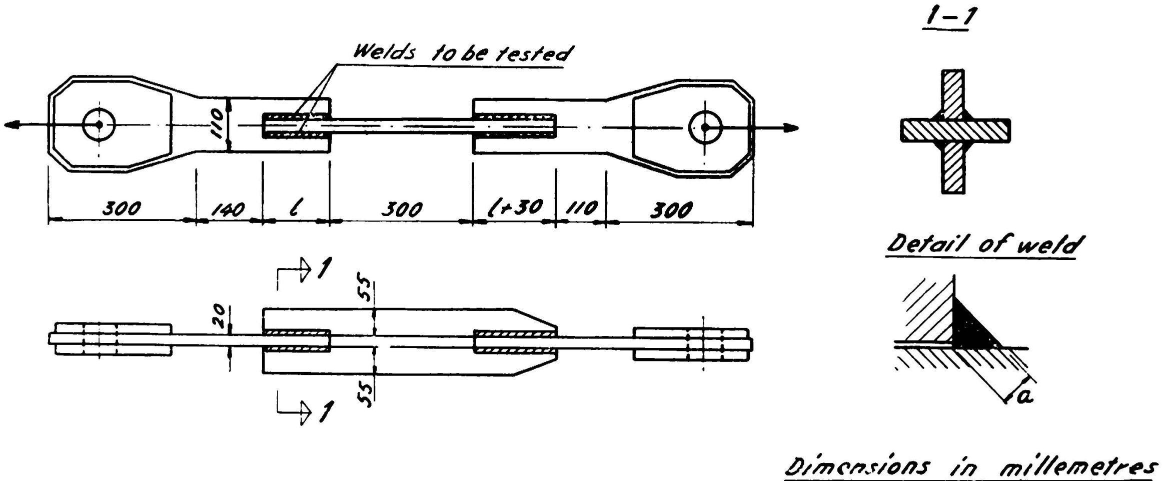

FIG. 3

The material for the test specimens was cut out of the same steel plate. From the tests made on sample bars which were cut out of the plate at the same time it was found that the yield point stress of the steel was 3550 kg per cm^2 and that its ultimate strength was 5350 kg per cm^2. The determination of the modulus of elasticity showed it to be 2.05 . 10^6 kg per cm^2. The welds were made by means of ASEA Z4P electrodes. The manufacturers state that the ultimate strength of this weld metal varies from 5000 to 5600 kg per cm^2 and that its ultimate elongation in a gauge length equal to five times the diameter of the test bar varies from 25 to 30 per cent.

TABLE 2

Length and Thickness of Fillet Welds on Test Specimens.

Test specimen No.	AI	AII	BI	BII	CI	CII
Length of weld, mm... ...	70	70	140	140	210	210
Thickness of weld, *a* mm...	3.5	6.0	3.0	6.0	3.5	6.0

The shearing stress distribution in the welds was determined by measuring the strains in one of the bars attached to the «connection plate». The strains were measured by means of resistant wire strain gauges (6 mm in gauge length), which were located in several gauge sections at right angles to the longitudinal direction of the bar. Four wire strain gauges were fitted on each side of the bar in each section. The location of the wire strain gauges is shown in Fig. 4. The wire strain gauges, which were opposite to each other on the two sides of the bar, were connected in series, so that direct readings of the average strain were taken at each point. Furthermore, wire strain gauges were also located in the central section in the free part of the bar in order that it might be possible to find out whether there was any appreciable amount of oblique tension.

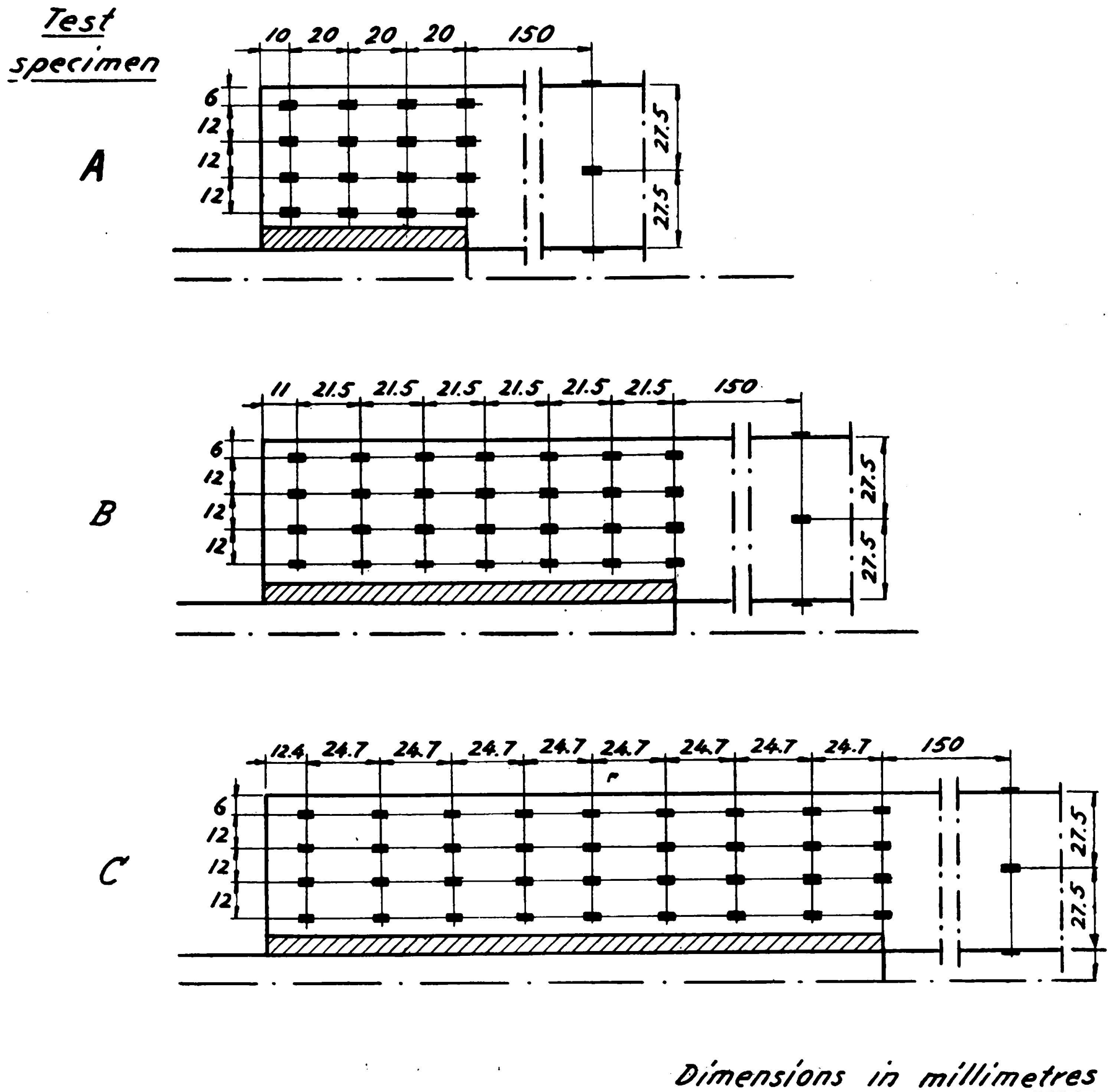

FIG. 4

If the stresses at the gauge points are known, then we can calculate the total tensile force D in each gauge section of the bar. If the direction x denotes the longitudinal direction of the bar, then the mean shearing stress in the two fillet welds of the bar is

$$\tau = \frac{1}{2a} \cdot \frac{dD}{dx} \tag{1}$$

Now it was the strains, and not the stresses, that were measured in the tests. The relation between stress and strain is

$$\sigma_x = E\,\varepsilon_x + \nu\,\sigma_y$$

In this relation, σ_y is unknown. However, an approximate correction can be obtained by means of the conditions for equilibrium, so that Eq. (1) can be written in the form

$$\tau = \frac{1}{2a}\left[\frac{dD_l}{dx} + \frac{1}{6}\nu H^2 \frac{d^3}{dx^3}(\mu D_l)\right]$$

where D_l is the nominal tensile force in a gauge section. This force is calculated from E and ε_x, but without taking σ_y into account, i. e.

$D_l = Et\int \varepsilon_x\,dy$. Furthermore, t is the thickness of the bar, and μ is a coefficient which is dependent on the distribution of ε_x over the cross section (for a uniformly distributed strain, μ is equal to unity). Poisson's ratio is put equal to 0.3 in what follows. The determination of the correction term is of course very uncertain, but, on the other hand, this term is relatively small.

The tests were made in a tension testing machine having a capacity of 100 tons. The strains were observed several times during each test. Only three of the test specimens, viz., AI, AII, and BI, could be tested to failure.

The ultimate loads observed in the tests are given in Table 3, together with the corresponding mean shearing stresses.

TABLE 3

Ultimate Loads

Test Specimen No.	Ultimate Load Tons	Mean Ultimate Shearing Stress kg per cm²
AI	40	4070
AII	58	3440
BI	85	5060

It is remarkable that the lowest mean shearing stress was observed in a short, thick weld (AII, $l = 70$ mm, $a = 6.0$ mm), while the greatest mean shearing stress occurred in a long, thin weld (BI, $l = 140$ mm, $a = 3.0$ mm). All the same, since only one specimen of each type was subjected to the tests, it is not possible to draw any reliable conclusions from the test results. All three test specimens that failed were fractured in the weld. The test specimen BI after failure is shown in Fig. 5.

Fig. 6 represents the shearing stress distributions corresponding to different loads and determined in conformity with the principle stated in the above. Each diagram shows the mean value of the observed values for each part of the weld between two gauge sections. The right-hand scale expresses the shearing stresses in the welds. The left-hand scale indicates the shearing force in two welds per centimetre of weld length ($2\ a\ \tau$). The actual shearing stress curve is of course continuous, but no attempt was made to draw this curve, since its details at the ends of the weld are unknown.

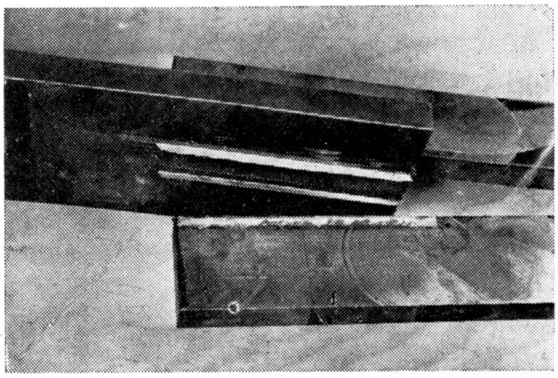

Fig. 5

The results shown in Fig. 6 are uncertain in some respects. These uncertainties are due to several causes, e. g. the fact that it was not possible to measure the strains at that edge of the bar which was welded.

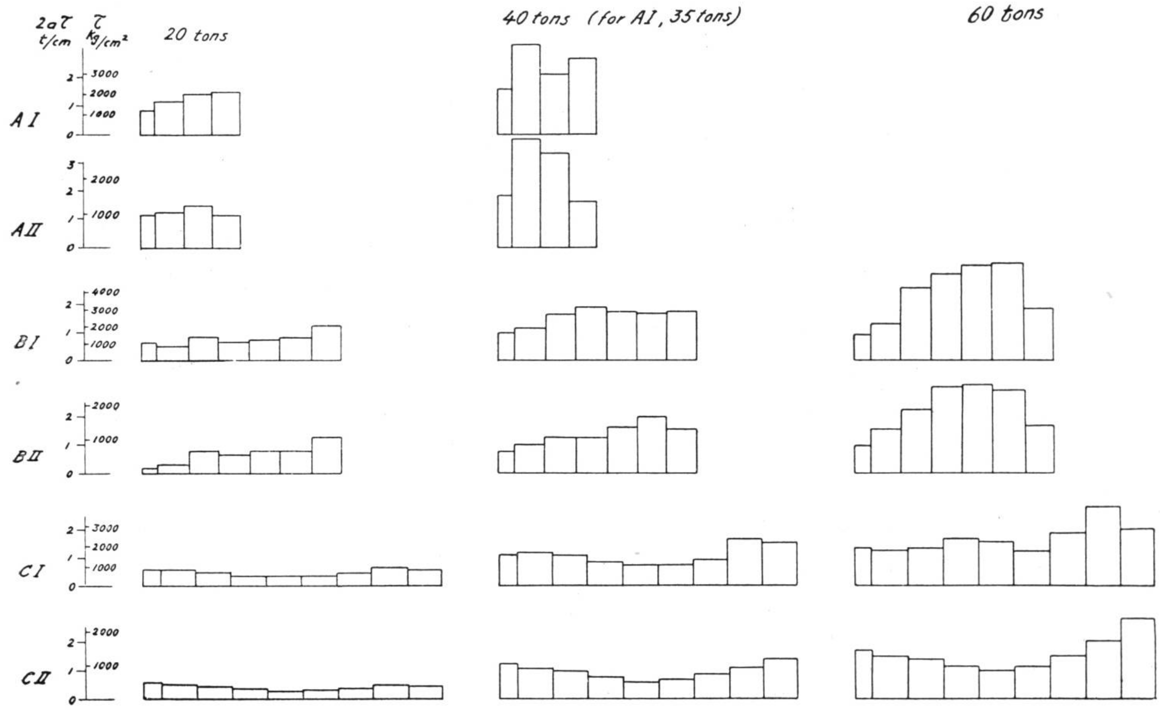

Fig. 6

Moreover, since the shearing stress is mostly dependent on the difference between the strains observed in two gauge sections, a relatively slight error in strain can give rise to a considerable error in the shape of the τ curve. The curves relating to the shortest welds are particularly sensitive to errors in measurements. Nevertheless, we can distinguish certain characteristic features in the shape of the curves. In the longest welds (210 mm), the shearing stress is greatest at both ends of the weld. The curve referring to the test specimen CI under a load of 60 tons is more irregular than the other curves. The curves relating to the medium long welds (140 mm) show a shearing stress distribution which is in part comparatively irregular under the lowest two loads, and which does not exhibit any marked concentrations of shearing stresses at the ends. Under the greatest load, the highest shearing stresses in these test specimens occur in the central portion of the weld. In the shortest welds (70 mm), the shearing stress distribution under the smallest load is fairly uniform. At greater loads, the highest shearing stresses are found in the central portion of the weld on the test specimen AII, while the shearing stress distribution relating to the test specimen AI is more irregular. The test results do not indicate any substancial difference between the test specimens with thinner and thicker welds. To sum up the inferences which may be drawn from the tests described in the above, it can be stated that the shearing stresses in long welds tend to be concentrated to a certain extent at the ends of the weld, while shorter welds exhibit a more uniform shearing stress distribution or in certain cases a concentration of the shearing stresses in the central portion of the welds. The terms «long» and «short» welds, as used in this connection, mean that the length of the weld is brought into relation with the dimensions of the bar attached to the connection plate.

ACKNOWLEDGEMENTS

The tests were made as graduation theses at the Royal Institute of Technology, Stockolm, Sweden. The first test series was carried out by Messrs C. Anger and B. Sjölander, while the second test series was performed by Messrs B. Alte and K. Rendel. The Authors wish to thank them for their work.

SUMMARY

This paper gives an account of an experimental investigation made in order to study the transmission of forces from a bar to a connection plate through fillet welds submitted to shear. The investigation comprised two series of tests.

The first test series dealt with the variation in the ultimate load with the design of the bars attached to the connection plate. All test specimens were provided with identical welds, but the cross sectional shape of the bars was varied in such a manner that different values of the stress $\sigma_{\perp}$ at right angles to the longitudinal direction of the weld and the stress $\sigma_{\parallel}$ parallel to this direction were to be expected. However, no appreciable difference in the ultimate loads corresponding to the various types of specimens was observed in the tests. Thus, it appeared that the effects produced by the stresses $\sigma_{\parallel}$ and $\sigma_{\perp}$ on the load-bearing capacity of the welds were of minor importance in this case.

In the second test series, the shearing stress distribution along the fillet welds was studied by means of strain measurements at three different lengths of weld and at two different thicknesses of weld. The results of these measurements indicate that the shearing stresses in long welds are concentrated to a certain extent at the ends of the welds, while shorter welds exhibit a more uniform shearing stress distribution or in certain cases a concentration of the shearing stresses in the central portion of the welds.

In all tests the mean value of the shearing stress at failure was not inconsiderably greater than $\frac{1}{\sqrt{3}}$ times the ultimate strength of the weld metal.

In so far as it may be considered justifiable to draw any conclusions from these tests, which comprised a very small number of specimens, all test results seem to indicate that it seems to be reasonable to make the following two assumptions in the design of fillet welds of the type illustrated in Fig. 1, viz., first, that the effects of the stresses $\sigma_{\parallel}$ and $\sigma_{\perp}$ may be disregarded, and second, that the distribution of the shearing stresses is fairly uniform even in comparatively long welds.

ZUSAMMENFASSUNG

Der Beitrag gibt die Ergebnisse experimenteller Untersuchungen über die Kraftübertragung von einem Stab in eine Verbindungsplatte durch schubbeanspruchte Schweissnähte. Dabei handelt es sich um zwei Prüfungsreihen.

Vorerst wurde die Abhängigkeit der Bruchlast vom Querschnitt des mit der Platte verbundenen Stabes untersucht. Dazu erhielten sämtliche Prüfstücke gleiche Schweissnähte, aber die Querschnittsform der Stäbe wurde so verändert, dass verschiedene Spannungen $\sigma_{\perp}$ senkrecht zur Längsrichtung der Naht und verschiedene Spannungen $\sigma_{\parallel}$ parallel zu dieser Richtung zu erwarten waren. Es liessen sich dann aber keine merklichen Unterschiede in den Bruchlasten der verschiedenen Prüfstücktypen feststellen. Damit scheint es, dass die Einwirkung der Spannungen $\sigma_{\parallel}$ und $\sigma_{\perp}$ auf das Tragvermögen der Nähte in diesem Fall unbedeutend ist.

In der zweiten Prüfungsreihe wurde die Schubspannungsverteilung entlang der Kehlnaht mittels Spannungsmessung an Schweissnähten von 3 verschiedenen Längen und 2 verschiedenen Dicken untersucht. Diese Messungen beweisen, dass in langen Nähten die Schubspannungen sich bis zu einem gewissen Mass in den Nahtenden anhäufen, während kürzere Nähte eine gleichmässige Verteilung oder in gewissen Fällen sogar eine Anhäufung der Schubspannungen ins Mittelteil der Schweissnaht aufweisen.

In allen Prüfungen war der mittlere Wert der Schubspannung beim Bruch nicht unbeträchtlich höher als $\frac{1}{\sqrt{3}}$ mal die Bruchfestigkeit des Schweissmetalls. Soweit es gerechtfertigt erscheint, aus diesen Prüfungen, welche nur eine geringe Anzahl von Untersuchungen umfassen, Schlussfolgerungen zu ziehen, so scheinen alle Ergebnisse derselben die Annahme zu bestätigen, dass:

1. die Einwirkung der Spannungen $\sigma_{//}$ und $\sigma_{\perp}$ als umbedeutend zu betrachten ist;
2. die Verteilung der Bruchlasten bei verhältnismässig langen Schweissnähten eine ziemlich gleichmässige ist.

RESUMO

O autor relata os ensaios a que se procedeu para estudar a maneira como se transmitem os esforços de uma viga a uma chapa de ligação, por intermédio de cordões de canto trabalhando ao corte. Efectuaram-se duas séries de ensaios.

Na primeira série determinou-se a variação da carga de rotura em função da secção das vigas soldadas à chapa de ligação. Em todos estes ensaios os cordões eram idênticos, mas a secção transversal das vigas era variável de modo a obter diferentes valores para as tensões $\sigma_{//}$ e $\sigma_{\perp}$, respectivamente paralela e perpendicular ao eixo longitudinal do cordão. Não se notou, no entanto, diferença apreciável entre as cargas de rotura correspondentes aos diversos ensaios. Parece portanto que o efeito das tensões $\sigma_{//}$ e $\sigma_{\perp}$ sobre a capacidade de carga das soldaduras é de pouca importância neste caso.

Na segunda série de ensaios estudou-se a distribuição das tensões de corte ao longo das soldaduras de canto, medindo as deformações para três comprimentos e duas larguras de cordão diferentes. O resultado destas medições mostra que para cordões compridos as tensões de corte estão em parte concentradas nas extremidades e que, para cordões mais curtos, a distribuição é mais uniforme ou apresenta, em certos casos, uma concentração de tensões na parte central do cordão.

Em todos os ensaios, o valor médio da tensão de corte correspondente à rotura, nunca ultrapassou muito $\frac{1}{\sqrt{3}}$ vezes a tensão de rotura do metal da soldadura.

Admitindo que se possam tirar conclusões deste estudo, que compreendeu um número muito reduzido de ensaios, os resultados parecem mostrar que, para o cálculo de cordões de canto do tipo indicado na fig. 1, é razoável admitir que:

1.° O efeito das tensões $\sigma_{//}$ e $\sigma_{\perp}$ é desprezível;

2.° A distribuição das tensões de corte é aproximadamente uniforme mesmo nos cordões relativamente compridos.

RÉSUMÉ

L'auteur décrit des essais effectués dans le but d'étudier le mode de transmission des efforts d'une barre à un gousset, par l'intermédiaire de cordons d'angle travaillant au cisaillement. L'ensemble de l'étude comprenait deux séries d'essais.

Dans une première série on a determiné la variation de la charge de rupture en fonction de la section des barres soudées au gousset. Dans tous ces essais les cordons étaient identiques, mais la section transversale des valeurs différentes pour les contraintes $\sigma_{//}$ et $\sigma_{\perp}$, respectivement parallèle et perpendiculaire à l'axe longitudinal du cordon. On n'a cependant pas remarqué de différence appreciable entre les charges de rupture correspondant aux divers essais. Il semble donc que l'effet des contraintes $\sigma_{//}$ et $\sigma_{\perp}$ sur la capacité de charge des soudures est peu important dans ce cas.

Dans une deuxième série d'essais on a étudié la répartition des contraintes de cisaillement le long des soudures d'angle, en mesurant les déformations pour trois longueurs et deux épaisseurs différentes de cordon. Le résultat de ces mesures montre que pour les cordons longs, les contraintes de cisaillement se concentrent, dans une certaine mesure, aux extrémités et que pour des cordons plus courts, la distribution est plus uniforme ou présente, dans certains cas, une concentration de contraintes dans la partie centrale du cordon.

Dans tous les essais, la valeur moyenne de la contrainte de cisaillement correspondant à la rupture, n'a guère dépassé de beaucoup $\frac{1}{\sqrt{3}}$ fois la contrainte de rupture du métal d'apport.

Pour autant que l'on puisse tirer des conclusions de cette étude qui n'a porté que sur un nombre très réduit d'essais, les résultats semblent montrer que, pour le calcul de cordons d'angle du type indiqué dans la fig. 1 il est raisonnable d'admettre que:

1° L'effet des contraintes $\sigma_{//}$ et $\sigma_{\perp}$ est négligeable;

2° La répartition des contraintes de cisaillement est approximativement uniforme, même pour des cordons relativement longs.

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