FAILURE OF WEB-TO-FLANGE WELDS OF BEAMS SUBJECT TO STATIC LOADS

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Introduction

During the latest two decades, spread of the welded structures widened out the research work concerning the load capacity of welded connections, leading to a number of scientific results in the domains of both statically loaded welds, and welds subject to fatigue loads.

In connection with the investigation of the load capacity of statically loaded welds, specialists are particularly interested in the laws of the failure of longitudinal fillet welds. Tests by WÄSTLUND and ÖSTLUND [1], FALTUS [2]

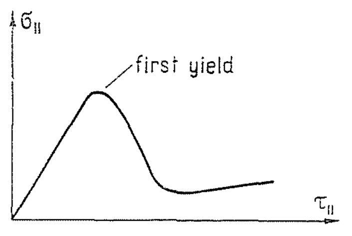

Fig. 1

and others proved that the ultimate strength of such welds was little affected by the stress component σ_{II}. The phenomenon has theoretically been explained by GÁLLIK [3]. He pointed out that stress component σ_{II} might linearly increase with the load not longer than up to the yield beginning in the weld. The yield abruptly reduces the value of the stress component σ_{II} and hereafter the stress component σ_{II} remains negligible beside the stress component τ_{II} (Fig. 1).

After longitudinal fillet welds connecting bars in tension or in compression, as a matter of course, researchers turned their attention to one type of the welds used the most frequently, this being the web-to-flange weld of plate girders. The question arose whether the stress component σ_{II} was important or negligible for the failure of web-to-flange welds. This problem has been experimentally studied in a number of laboratories. Tests by NEUMANN [4] and

GÁLLIK [3] unambiguously proved that the stress component was unimportant for the failure of the web-to-flange weld of steel beams under static loads too. The tests even pointed out that web-to-flange welds of steel beams subject to static load practically cannot fail. Web-to-flange welds in the test beams remained usually undamaged even after the complete failure of the beams, though the welds have deliberately been made too weak (with a small cross-sectional area). In this paper a hypothesis and a test are briefly described, likely to furnish satisfactory explanation to the phenomenon mentioned above.

Hypothesis concerning the flange displacement

From the fact that not even small web-to-flange welds of steel beams subject to static load fail, one might conclude that in case of such welds not only stress component $\sigma_{||}$ *tends to zero in the failure zone but also stress component* $\tau_{||}$ *lags significantly behind the load, instead of directly increasing with it. If this is true, it can only be attributed to the considerable displacement of the flange, due to the plastic deformation of the weld material.*

The relative displacement of the connected parts of steel beams is verified by a number of tests. For example, BRYLA and CHMIELOWIEC [5] pointed out as early as in the 1930's that the load capacity of steel beams with welded or riveted joints was lower than that of rolled beams of the same proportions but without joints. Great flange displacements appeared from load test results, some years ago, of the Elisabeth-bridge across the Danube in Budapest [6].

The relative displacement of beam flanges is an old problem for the researchers. A number of papers have been published in recent decades dealing with this question [7]. The investigations were limited, however, to the elastic range and commonly nothing else than elastic flange displacements to be negligible was stated. The question, how much the flange displaced after the yield of the web-to-flange weld and how this reacted on stresses, was usually ignored.

It remains certain that the flange displacement due to the yield of the web-to-flange weld is greater by about one order of magnitude than the elastic one and thus, the formula

$$\tau_{||} = \frac{T \cdot S}{J \cdot v}$$

used for the determination of the stress component $\tau_{||}$ of the web-to-flange weld, after its plastic deformation, is not valid any more. In this case, the formula gives only the theoretical upper limit of the stress component $\tau_{||}$. The real value of the stress component $\tau_{||}$ is always lower than this latter. According to the theoretical considerations the difference between the theoretical limit and the real stress component $\tau_{||}$ is the more significant, the greater is the plastic deformability of the weld material and the shorter is the sheared length of weld, that is, the span of the beam. At the same time, this means that the design application of this formula representing an upper limit results in a higher safety against failure than expected. This may first of all be observed for highly ductile steel welds and for beams of short span (or under two symmetrical loads).

Failure tests

In 1966, this problem has been studied experimentally in the laboratory of the Department of Steel Structures of the Budapest Technical University. In planning the tests, starting requirement was to have a specimen material with mean plastic properties, since else *either absence of the shear stress increase remains unobserved* (the material is too stiff), *or no shear failure will come about* (the material is too plastic).

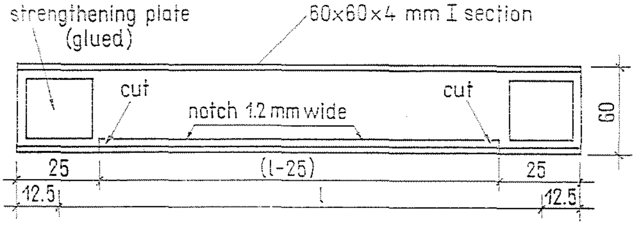

Fig. 2

Taking these viewponts into consideration, the specimens have been made of an alloy named HEGAL 34 (AlMgZnTi) of which a pressed I-section of 60 × 60 × 4 mm was available. Beams of given length have been cut off of this material. On both sides of the test beams, to 6,2 mm from the edges of the lower flange, grooves 1,2 mm thick have been cut out with a slitting saw to weaken the web. The groove was cut out along the whole length of the beam except a portion of 2,5 cm on both ends. Here, grooves seemed to be undesirable because of the risk of a destructive compression or buckling in the thin part of the web, due to the relatively great support reactions. In order to avoid disturbances owing to the thicker portion of the web, over the supports, at the ends of the grooves the lower flange has been cut through up to the upper edge of the grooves (to provide for a stress transfer into the lower flange through the weakened web). To eliminate the risk of local buckling at the supports, plates have been glued up on both sides of the web (Fig. 2).

Reference test beams have been made of the alloys HEGAL 34 and MASZIL 28, *lower in strength but more ductile*, with and without grooves, respectively.

A testing machine WPM applied a single load P at midspan on beams supported by rollers on a bending table.

Five of the eight HEGAL 34 specimens failed by shear of the weakened web part starting from the support towards midspan (Fig. 3). The other three specimens failed by instability (i. e. lateral buckling) rather than by shear in

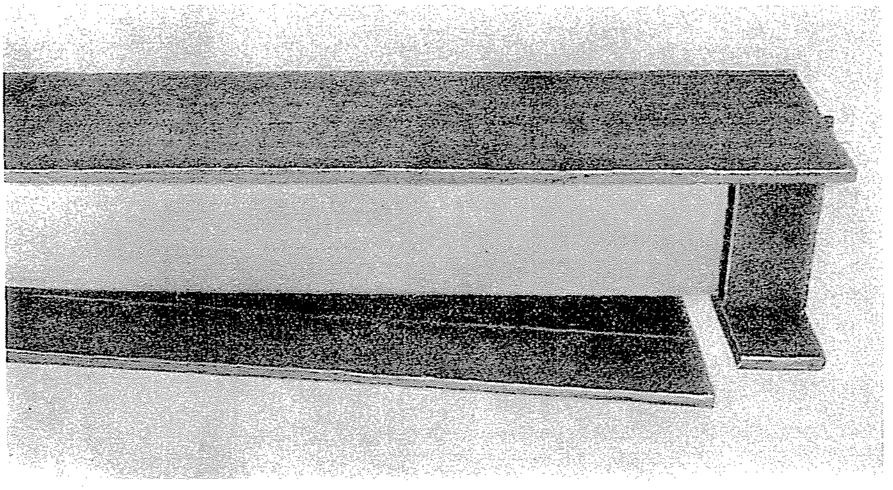

Fig. 3

the weakened parts of the web. One of them had a relatively large span (specimen 6), while webs of the other two specimens were not grooved (reference specimens No. 7 and 8). Of the four reference specimens of alloy MASZIL 28, the two short-span ones failed by shear, the other two by instability.

The ultimate strengths of the HEGAL 34 specimens are compiled in Table I. *Tabulated data unambiguously show the ultimate shear load on the fillet to increase with shorter spans.* The same is true for MASZIL 28 beams failed by shear.

Checking test results by computation

Test results have been checked by computation, based on the assumption that the formulae deduced for beams with elastic joints may be applied in the stage of failure, if the spring factor has been determined from a chord drawn to the end point of the characteristic curve of the material (method of the chord modulus). This method has previously been applied with favourable results, for example to compute the ultimate strength of longitudinal fillet welds and of glued connections [7], [8], [9].

In our case, the characteristic $\tau - \gamma$ curve of HEGAL 34 has been plotted on the basis of torsion tests. Tests showed an ultimate shear strength of 2,28 Mp/cm^2, involving a specific rotation of 0,28 radian. Thus, according to the chord modulus, the spring factor of the fillet part 1,2 mm high and 1,0 mm thick of the test beams was likely to be

$$c = \frac{2{,}28}{0{,}28} \cdot \frac{0{,}10}{0{,}12} = 6{,}8 \ \mathrm{Mp/cm^2}.$$

The calculation involved the differential equation

$$\frac{d^2N(x)}{dx^2} - \omega^2 \cdot N(x) + \Omega \cdot M_0(x) = 0$$

describing the stress pattern of simply supported composite beams with elastic connections, where:

$N(x)$ is the flange force; $M_0(x)$ is the moment from the external forces;

ω and Ω are constants depending on the cross-section and the spring factor, respectively [10].

For the test beams we obtained:

$$\omega = 0{,}147 \ \frac{1}{\mathrm{cm}},$$

$$\Omega = 0{,}0035 \frac{1}{\mathrm{cm}^3}$$

From the solution of the differential equation the ultimate strength could be re-calculated:

$$P_t = 2v \frac{J}{S} \cdot \tau_B \cdot \frac{ch\omega \dfrac{l}{2}}{ch\omega \dfrac{l}{2} - 1}$$

In the formula:

v — thickness of the weakened web portion;
J — moment of inertia of the cross-section;
S — static moment about the flange section centroid;
τ_B — shear strength of the material; and
l — beam span.

Table I

No. of Specimen	Span l [cm]	Width of neck v [mm]	Failure load P_f [Mp]		
			Test	Theoretical	
				A	B
1	22.5	1.0	4.30	4.45	4.88
2	32.5	1.0	4.12	4.45	4.88
3	42.5	1.0	3.15	3.08	3.25
4	52.5	1.0	3.02	2.92	2.96
5	57.5	1.0	2.80	2.88	2.78
6*	62.5	1.0	2.63	2.86	2.76
7*	22.5	3.0	—	—	—
8*	57.5	3.0	—	—	—

* Failure due to instability.

The re-calculated ultimate strength values appear in column A of Table I. Measured and calculated ultimate strength values show a good agreement (a major difference appears for specimen No. 2 alone). The same values are compared in Fig. 4. The straight dotted line represents the lower limit of the theoretical ultimate strength values, it being the asymptote of the theoretical ultimate strength curve.

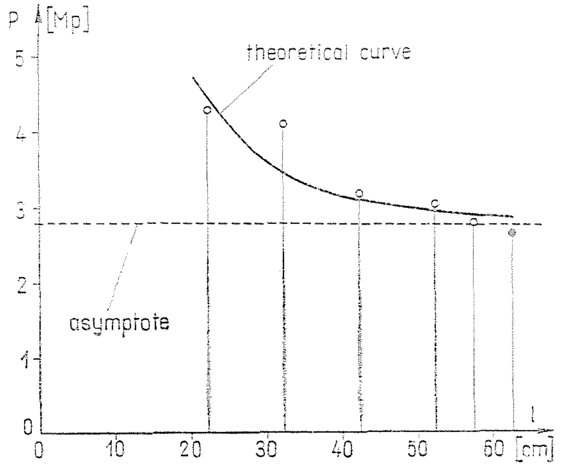

Fig. 4

In addition, a simpler method has been worked out to calculate ultimate strength values. This latter is based on the same assumption as the differential equation above, but its solution is based on the Ritz-Timoshenko energy method. The final formulae are relatively simple and are of the same form as those for rigidly connected beams, only the moment of inertia J has to be replaced by a theoretical moment of inertia J_t, also depending on the spring factor and on the span [7]. The obtained ultimate strength values may be seen in column B of Table I.

Conclusions

The assumption of flange displacement in the stage of failure due to plastic deformation of the web-to-flange weld has been substantiated by the described tests and computations. The following conclusions concerning the web-to-flange welds subject to static loads can be drawn:

a) Actual formulae applied for designing web-to-flange welds are mostly inadequate to estimate the safety against failure.

b) Besides of the strength of the weld material and the beam geometry, the ultimate load of web-to-flange welds depends also on the plastic properties of the weld. The greater the plastic deformability of the weld and the shorter the beam span, the higher is the safety against failure of web-to-flange welds designed by conventional formulae.

Summary

Web-to-flange welds of steel beams subject to flexural and shear stresses do not, in general, fail under the effect of static loads. This can be attributed to flange displacement prior to failure, owing to the plastic deformation of the web-to-flange weld, the displacement affecting favourably its stress state. This assumption was justified by tests and computations presented in the paper. For a better evaluation of the test results, specimens made of aluminium alloys have been applied, exhibiting the same phenomenon but more ready to test than steel specimens because of their rather unfavourable plastic properties.

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