Lecture 17

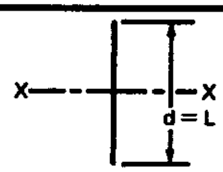
Formulas for Calculating Force per Unit Length on Welds

Type of Loading	Standard Formula for Unit Stress	Formula for Force per Unit Length
Tension or compression	$\sigma = \frac{P}{A}$	$f = \frac{P}{L_w}$
Vertical shear	$\tau = \frac{V}{A}$	$f = \frac{V}{L_w}$
Bending	$\sigma = \frac{M}{S} = \frac{Mc}{I}$	$f = \frac{M}{S_w} = \frac{Mc}{I_w}$
Torsion	$\tau = \frac{Tc}{J}$	$f = \frac{Tc}{J_w}$

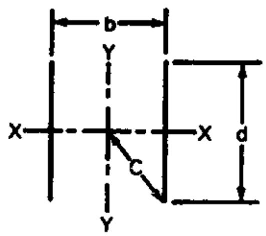
σ = normal stress
τ = shear stress
f = force per unit length
P = concentrated load
V = vertical shear load
A = total area of cross section
L_w = total length of a line weld
c = distance from neutral axis to the extreme fibers of a line weld
T = torque on the weld joint
S = sections modulus of an area =
S_w = section modulus of a line weld
J = polar mement of inertia of an area
J_w = polar moment of inertia of a line weld
I = moment of inertia
I_w = moment of inertia of a line weld

Lecture 17

Table 5.12
Properties of Welded Connections Treated as a Line



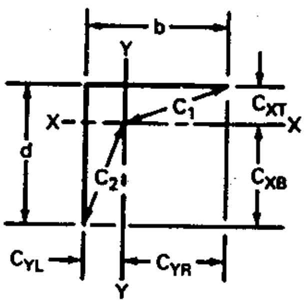
$$I_x = \frac{d^3}{12} \qquad S_x = \frac{d^2}{6}$$



L = 2d

$$I_x = \frac{d^3}{6} \qquad S_x = \frac{d^2}{3} \qquad J_w = \frac{d}{6}(3b^2 + d^2)$$

$$I_y = \frac{b^2 d}{2} \qquad S_y = bd \qquad C = \frac{(b^2 + d^2)^{1/2}}{2}$$



L = b + d

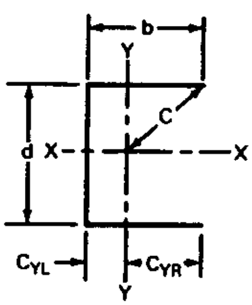
$$I_x = \frac{d^3}{12}\left(\frac{4b + d}{b + d}\right) \qquad S_{XT} = \frac{d}{6}(4b + d) \qquad S_{XB} = \frac{d^2}{6}\left(\frac{4b + d}{2b + d}\right)$$

$$I_y = \frac{b3}{12}\left(\frac{b + 4d}{b + d}\right) \qquad S_{YL} = \frac{b}{6}(b + 4d) \qquad S_{YR} = \frac{b^2}{6}\left(\frac{b + 4d}{2d + b}\right)$$

$$J_w = \frac{b^3 + d^3}{12} + \frac{bd\,(b^2 + d^2)}{4\,(b + d)}$$

$$C_{XT} = \frac{d^2}{2(b + d)} \qquad C_{XB} = \frac{d}{2}\left(\frac{2b + d}{b + d}\right) \qquad C_1 = (C_{XT}^2 + C_{YR}^2)^{1/2}$$

$$C_{YL} = \frac{b^2}{2(b + d)} \qquad C_{YR} = \frac{b}{2}\left(\frac{b + 2d}{b + d}\right) \qquad C_2 = (C_{XB}^2 + C_{YL}^2)^{1/2}$$



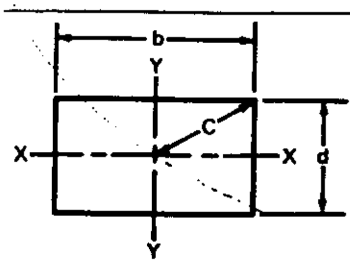
L = 2b + d

$$I_x = \frac{d^2}{12}(6b + d) \qquad S_x = \frac{d}{6}(6b + d)$$

$$I_y = \frac{b^3}{3}\left(\frac{b + 2d}{2b + d}\right) \qquad S_{YL} = \frac{b}{3}(b + 2d)$$

$$C_{YL} = \frac{b^2}{2b + d} \qquad C_{YR} = \frac{b(b + d)}{2b + d} \qquad S_{YR} = \frac{b^2}{3}\left(\frac{b + 2d}{b + d}\right)$$

$$C = \left[C_{YR}^2 + \left(\frac{d}{2}\right)^2\right]^{1/2} \qquad J_w = \frac{b^3}{3}\left(\frac{b + 2d}{2b + d}\right) + \frac{d^2}{12}(6b + d)$$



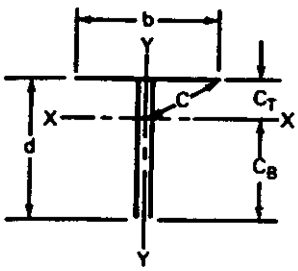
L = 2 (b + d)

$$I_x = \frac{d^2}{6}(3b + d) \qquad S_x = \frac{d}{3}(3b + d)$$

$$I_x = \frac{d^2}{6}(b + 3d) \qquad S_y = \frac{b}{3}(b + 3d)$$

$$J_w = \frac{(b + d)^3}{6} \qquad C = \frac{(b^2 + d^2)^{1/2}}{2}$$

Table 5.12 (Continued)



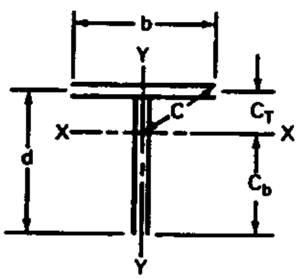
L = b + 2d

$$I_x = \frac{d^3}{3}\left(\frac{2b + d}{b + 2d}\right) \qquad S_{XT} = \frac{d}{3}(2b + d) \qquad S_{XB} = \frac{d^2}{3}\left(\frac{2b + d}{b + d}\right)$$

$$I_Y = \frac{b^3}{12} \qquad S_Y = \frac{b^2}{6} \qquad C_T = \frac{d^2}{b + 2d}$$

$$J_W = \frac{d^3}{3}\left(\frac{2b + d}{b + 2d}\right) + \frac{b^3}{12} \qquad C_b = d\left(\frac{b + d}{b + 2d}\right)$$

$$C = \left[C_T^2 + \left(\frac{b}{2}\right)^2\right]^{1/2}$$



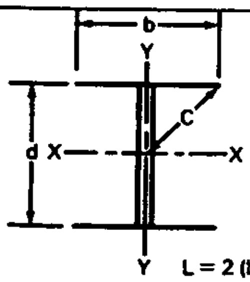
L = 2 (b + d)

$$I_x = \frac{d^3}{6}\left(\frac{4b + d}{b + d}\right) \qquad S_{XT} = \frac{d}{3}(4b + d) \qquad S_{XB} = \frac{d^2}{3}\left(\frac{4b + d}{b + d}\right)$$

$$I_Y = \frac{b^3}{6} \qquad S_Y = \frac{b^2}{3} \qquad C_T = \frac{d^2}{2(b + d)}$$

$$C_B = \frac{d}{2}\left(\frac{2b + d}{b + d}\right)$$

$$J_W = \frac{d^3}{6}\left(\frac{4b + d}{b + d}\right) + \frac{b^2}{6} \qquad C = \left[C_T^2 + \left(\frac{b}{2}\right)^2\right]^{1/2}$$

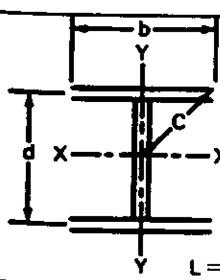


L = 2 (b + d)

$$I_x = \frac{d^2}{6}(3b + d) \qquad S_X = \frac{d}{3}(3b + d)$$

$$I_Y = \frac{b^3}{6} \qquad S_Y = \frac{b^2}{3}$$

$$J_W = \frac{d^2}{6}(3b + d) + \frac{b^3}{6} \qquad C = \frac{(b^2 + d^2)^{1/2}}{2}$$

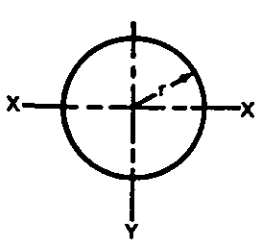


L = 2 (2b + d)

$$I_x = \frac{d^2}{6}(6b + d) \qquad S_X = \frac{d}{3}(6b + d)$$

$$I_Y = \frac{b^3}{3} \qquad S_Y = \frac{2}{3}\ b^2$$

$$J = \frac{d^2}{6}(6b + d) + \frac{b^3}{3} \qquad C = \frac{(b^2 + d^2)^{1/2}}{2}$$



L = 2πr

$$I = \pi r^3 \qquad S_W = \pi r^2 \qquad J_W = 2\pi r^3$$

Design Rules for Arc-Welded Connections in Steel Submitted to Static Loads

worked out by Commission XV-A
edited by D.K.Feder

Contents: Page

1. Preface

This document results from international co-operation over the past decade. Research was carried out in various countries, an International Test Series produced a range of valuable results, and the problems of the design rules were discussed at length as confirmed by the list of documents in Appendix 2.

The original plan and hope was to produce a detailed internationally accepted IIW standard for the calculation of welded connections. However, the discussions over many years in the commission and subcommission A have shown that such an aim is not realistic: the adherence to nationally accepted design rules is too strong. This document gives the basic principles for the calculation of welded connections. Whenever practicable it points out simplifications or refinements of the general formulae, but leaves the actual choice to the designer.

2. Range of application

These recommendations are intended for the calculation of statically loaded welded connections in carbon and lowalloy steels having a minimum tensile strength less than 600 N/mm^2, a ratio of yield to ultimate strength ≤ 0.8, and a minimum elongation $\varepsilon_{10} \geq 12$ %. The welds are supposed to be made by arc welding (i.e. coated electrode, gas shielded and submerged arc welding).

3. Workmanship and techniques

For welds to be calculated according to these rules the following conditions should be fulfilled:

3.1 A design that takes into account the particular characteristics of welded connections.

3.2 Correct choice of base and filler materials in respect ~~to~~ of
- the service and loading conditions of the connection
- the properties and thickness of the material
- the welding process and welding conditions

3.3 Fabrication and erection in accordance with drawings and specifications which indicate where necessary
- type and dimension of the weld
- limiting conditions for welding
- the welding process
- the welding sequence
- the type of filler materials
- extent and method of inspection
- acceptability of defects
- special requirements to which the connection shall conform.

3.4 A sound welded joint assured by adequate supervision of
- preparation and assembly for welding
- welding procedure
- welders skilled for the task

4. Calculation of groove welds in butt joints

4.1 Full penetration groove welds need not be analysed.

4.2 Where their use is permissible partial penetration groove welds (Fig. 1) are to be analysed as fillet welds.

groove weld = butt weld.

5. Definitions and general provisions concerning fillet welds

5.1 For calculation the thickness "a" of a fillet weld equals the height of the largest inscribed triangle (Fig. 2), provided there is an average root penetration in accordance with curve "b" in Fig. 3.

In the case of automatic submerged arc welding the weld thickness may be increased by 20 % to a maximum of 2 mm according to curve "c" in Fig. 3. In order to be able to assume a higher increase in weld thickness for automatic submerged arc welding, or increased weld thicknesses for other processes, the extent and uniformity of penetration must be demonstrated by tests.
On the other hand, for welds with insufficient root penetration, the thickness "a" has to be reduced accordingly, provided the lack of penetration is proved to be less than 0.1·a and less than 1 mm.

5.2 The throat section of a fillet weld is the plane through root and height (Fig. 4, see Fig. 2 for "height").

5.3 The surface "A" of the throat section is the product of weld thickness "a" and weld length "l".

The weld length used in the strength calculations is the theoretical length without reduction for head or end craters. different from IIS/IIW-139-64 (XV-156-63)

Welds with lengths shorter than 8 times the weld thickness "a" may not be utilized for the transmission of forces in the strength calculations.

6. Calculation of fillet welds

6.1 Strength calculation

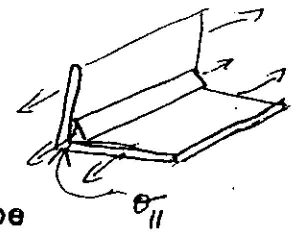
In order to calculate the strength of a weld the forces to be transmitted by this weld and the line of action of these forces must be determined by analysis either elastical or plastical. Sometimes in addition the distribution of the forces along the weld length has to be found, but generally the forces can be considered to be uniformly distributed along the throat section or subdivisions of it, provided the internal equilibrium is satisfied and the welds have sufficient deformation capacity (see also chapter 7).

6.2 General loading

If a weld or part of a weld is loaded in an arbitrary direction, and assuming that the forces are uniformly distributed over the length "l" of the weld, the stress caused by them can be resolved into components with respect to the throat section (see Fig. 4):

a normal stress ($\sigma_\perp$) perpendicular to the throat section, a shear stress ($\tau_{/\!/}$) acting in the throat section parallel to the axis of the weld and a shear stress ($\tau_\perp$) acting in the throat section transverse to the axis of the weld.

A normal stress $\sigma_{/\!/}$ along the axis of the weld need not be accounted for. 1)



Both normal and shear stresses are considered to be uniformly distributed across the thickness of the weld.

6.3 Allowable stresses and comparison stress

For the stresses $\sigma_\perp$, $\tau_{/\!/}$ and $\tau_\perp$ or a combination thereof it must be shown that they comply with the conditions

$$\beta \sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{/\!/}^2)} \leq \sigma_c \qquad (6.1)$$

and

$$\sigma_\perp \leq \sigma_c \qquad (6.2)$$

Provided the connection conforms to the requirements listed in section 3, the permissible comparison stress equals the permissible tensile stress $\bar{\sigma}_t$ in the base material and

Y.S. UTS

$\beta = 0.7$ for Fe E 240 (Fe 360) ordinary MS

$\beta = 0.85$ for Fe E 350 (Fe 510) HS structural steel

For other steel qualities β may be determined by means of linear interpolation proportional to the guaranteed yield strength of these steels.

Footnote reference, see Appendix 1

watch out, cause in exercises they

6.4 For the sake of simplicity of the design rules any approximation lying on the safe side of the ellipsoid described by the above formula may be adopted. [2] For typical cases of loading of fillet welds simple formulae can be derived, thus eliminating the necessity to solve the above expression (see Appendix 3).

7. Analysis of welded joints

7.1 In order to calculate the strength of a welded joint the load distribution among the individual welds composing the joint has to be determined. There are two approaches:

a) the load in the weld is assumed to depend directly on the stress in the adjacent parent material; Then match weld metal strength.

as, b) the joint is considered to constitute a separate unit in the structure and the loading of the individual welds is derived from the load of the entire joint.

The actual load distribution will fall between these two approaches. The approach which the designer chooses will depend on the type of structure, the type of joint, and the type of analysis, elastic or plastic. (Plastic sometimes easier)

7.2 Calculation based on the stress in the parent material

7.2.1 This approach is universal and is the way to design welds in complicated structural parts (see 7.3.1).

The stresses in the parent material may be determined by either elastic or plastic analysis.

7.2.2 When using plastic analysis a stress distribution is assumed which satisfies equilibrium conditions and where no stress exceeds the yield stress of the parent material.

In applying plastic analysis the welds must have sufficient deformation capacity. In general this is the case if the weld is designed for a stress equal to at least 0.7 of the yield stress of the parent material. [3]

Welds in plastic hinges should be able to transmit at least the full yield force of the connected parts.

7.2.3 A calculation based on the stresses in the parent material offers a very simple way to determine the required weld thickness "a" and this can be applied widely:

$$a = \frac{\overline{\sigma}_p}{\overline{\sigma}_w} t \qquad (7.1)$$

Equating forces.

where

$\overline{\sigma}_p$ = stress in parent material

t = thickness of parent material

$\overline{\sigma}_w$ = allowable stress in the weld according formula (6.1) and (6.2)

still gotta work out σ_w. Not ½ as simple as here in scheme

7.3 Calculation based on the load of the entire joint

7.3.1 In cases where the local stress distribution in the parent material adjacent to the welds is unknown or can be determined only by a laborious analysis, the welds may be designed by a plastic analysis of the complete joint. This is achieved by

1) resolving the loading of the joint into forces or groups of forces related to the pattern of welds in the joint,

2) assigning to the individual welds those forces which they are best suited to take according to their orientation,

3) assuming yielding in the single welds over their entire length.

This procedure is well illustrated by the examples given in paragraphs 7.3.2 to 7.3.5. Often there are several possibilities to resolve a given loading; all of them are valid if they fulfill equilibrium conditions and stresses nowhere exceed the yield stress.

Care has to be taken that the stiffness of the connected parts and the deformation capacity of the welds is in agreement with the assumptions made. Otherwise a reduced effective length must be used that has to be determined from rupture tests on the different types of connections.

7.3.2 The lap-joint loaded by a normal force

1) Side fillet welds only, see Fig. 5a

The total load taken by the connection is given by

$$P = \sum a \cdot l \cdot \overline{\sigma}_W \qquad (7.2)$$

When using long welds care has to be taken to assure sufficient deformation capacity by limiting the length to thickness ratio of the welds. [4)]

2) Side and end fillet welds, see Fig. 5b

The total load taken by the connection is given by

$$P = \sum (a \cdot l \cdot \overline{\sigma}_W) \qquad (7.3)$$

σ_W different for side & end fillets. Not as simple as it looks.

with different values of $\overline{\sigma}_W$ for side and end welds. The connection can also be designed by the simplified formula [5)]

Don't use unless weld geometry well proportioned.

$$P = 0.8 \sum a \cdot l \cdot \sigma_c \qquad (7.4)$$

? cf. v.d. Eb's approach

If different thicknesses of side and end fillet welds are feasible, the end welds should be made thicker because they have less deformation capacity than side welds, and deformation capacity increases with thickness.

3) Small eccentricities resulting from the section of the joined parts (Fig, 5c) may be neglected for the strength calculation of the welds. This does not mean that they can also be neglected for the members themselves.

7.3.3 The lap-joint loaded by a shear force and/or a moment

If only the welds $2 \times l_1$ and l_2 (see Fig. 6) are present, the most "natural" assumption for the load distribution among the welds is:

the shear force P is taken by weld $a_2\ l_2$ giving

Note diff.nt. assumptions from Germany.

$$\tau_{//} = P/a_2 \cdot l_2 \qquad (7.5)$$

the moment $M = P \cdot L$ is taken by the couple of welds $a_1\ l_1$ giving

$$\tau_{//} = \frac{M/l_2}{a_1 l_1} \qquad (7.6)$$

If also a weld l_3 is present the shear force could be assigned equally to welds l_2 and l_3, and the moment (now $M = P \cdot (L - l_1/2)$) to the welds l_1. However, if this assumption leads to large differences in weld thicknesses, an alternative assumption is to assign part of the moment to the couple of welds l_2 and l_3. On the other hand it would not be good practice to assign the shear force to the longitudinal welds l_1 since this assumption contradicts the shear distribution in the member.

7.3.4 The beam-column connection

The most common and reasonable way to design welds in a beam-column connection is to allocate the moment as a couple of forces to the welds at the flanges and the shear force to the web welds (see Fig. 7). However, this approach is possible only if the column is provided with stiffeners to feed the loads from the flanges into the column-web. Where there are no stiffeners (which may often be the more economic solution) the design has to be based on a reduced effective length of the weld:

$$b_{eff} = c_1 \cdot t_1 + 2 \cdot t_2 \quad (7.7)$$

(see Fig. 8 for t_1, t_2). The factor c_1 takes the following values: [6]

Takes account of different member stiffnesses on weld efficiency

c_1			
7	5	Tensile flange	Fe 360
10	7	compressive flange	
5	4	tensile flange	Fe 510
7	6	compressive flange	
I section Fig. 8a	box-section Fig. 8b		

The welds must be designed for a normal stress equal to at least 0.7 the yield stress of the adjacent parent material and the calculated weld thickness "a" must be applied over the full width "b".

7.3.5 Connections loaded in torsion

This is a less common form of loading where the axis of the moment is perpendicular to the plane of the connection (Fig. 9). It is advisable to design the welds for the appropriate type of torsional loading:

a) St. Venant torsion of open sections
b) warping torsion
c) torsion of closed sections.

Again not as easy as have one scheme, see last 'handout' for both demonstration.

Whilst cases a) and c) present no problem, since welds are loaded by $\tau_{//}$ only, the amount of warping torsion is often difficult to estimate because it depends on the restraints posed by the rest of the structure; in plastic design it can be neglected unless essential for equilibrium (flange bending). Designing the welds for 0.7 the yield stress of the adjacent parent material [3] will be the best solution in such cases.

8. Figures

FIG. 1

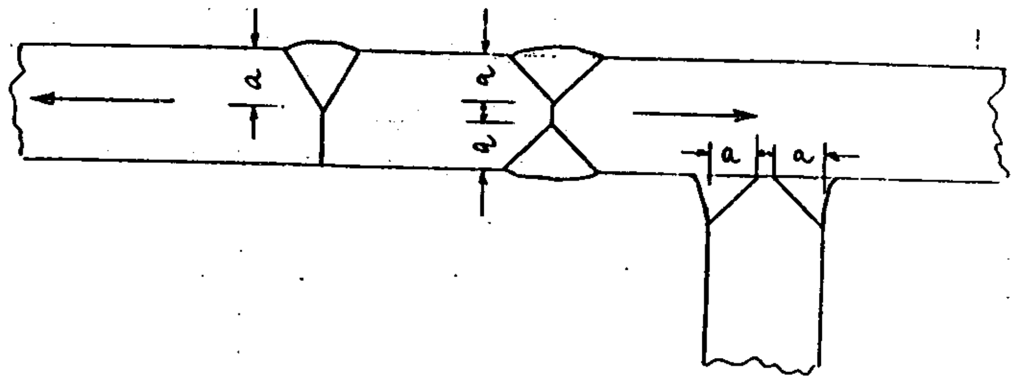


FIG. 2

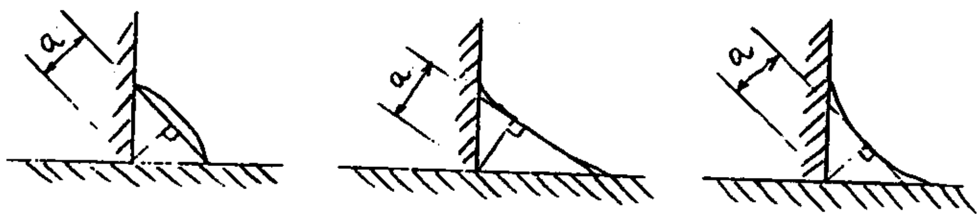
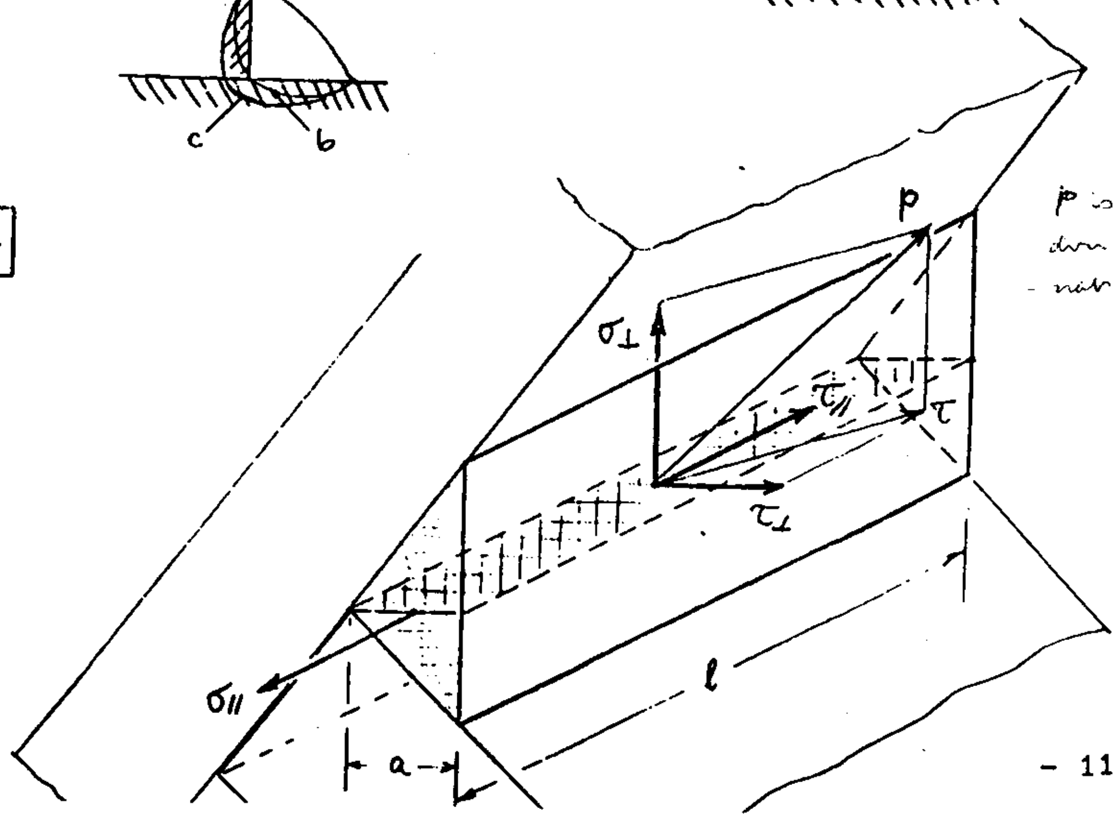


FIG. 3

FIG. 4



p is true dirn of stress – not component!

FIG. 5

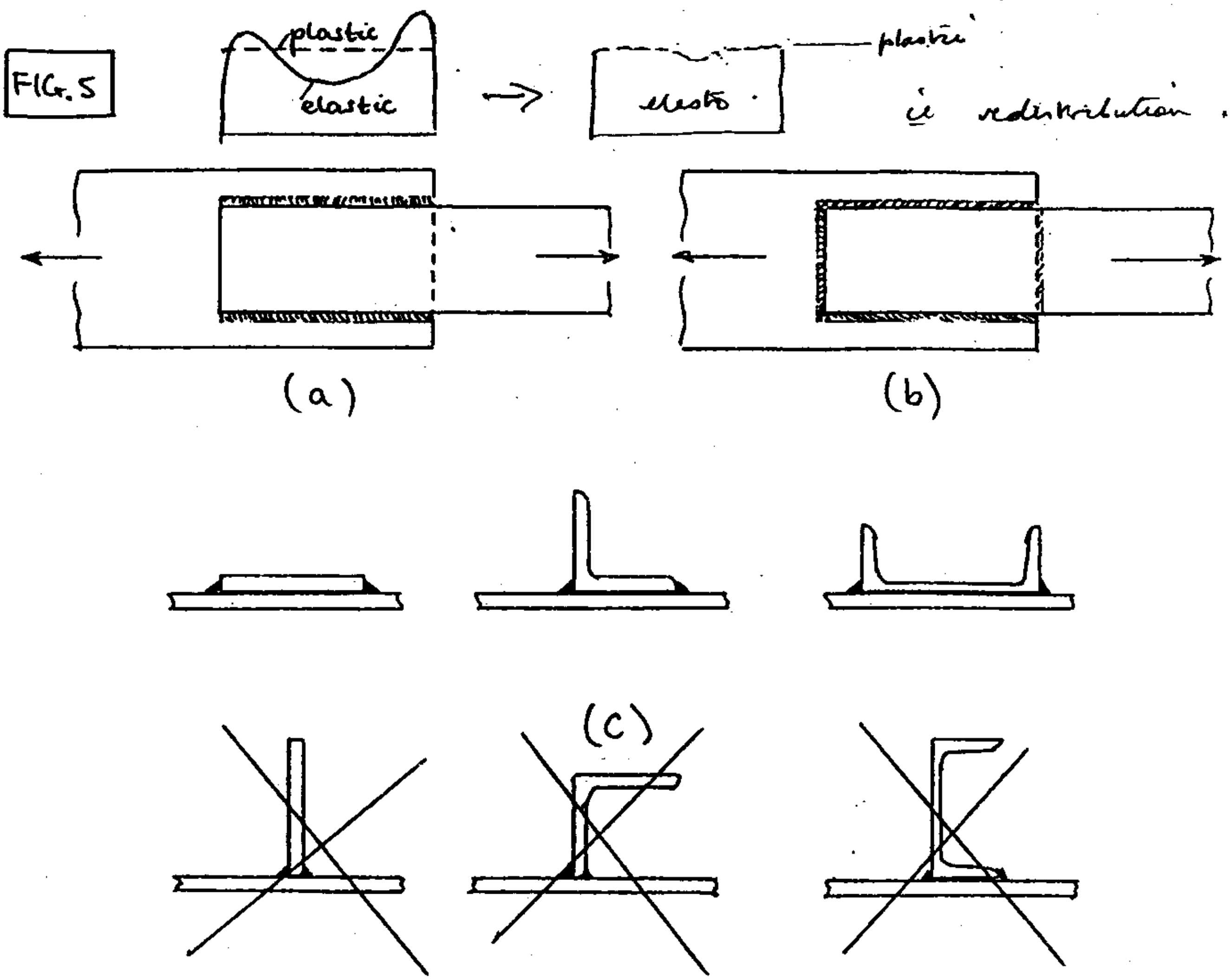


FIG. 6

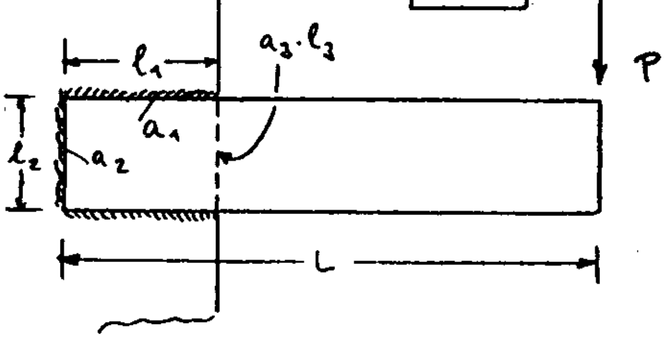


FIG. 7

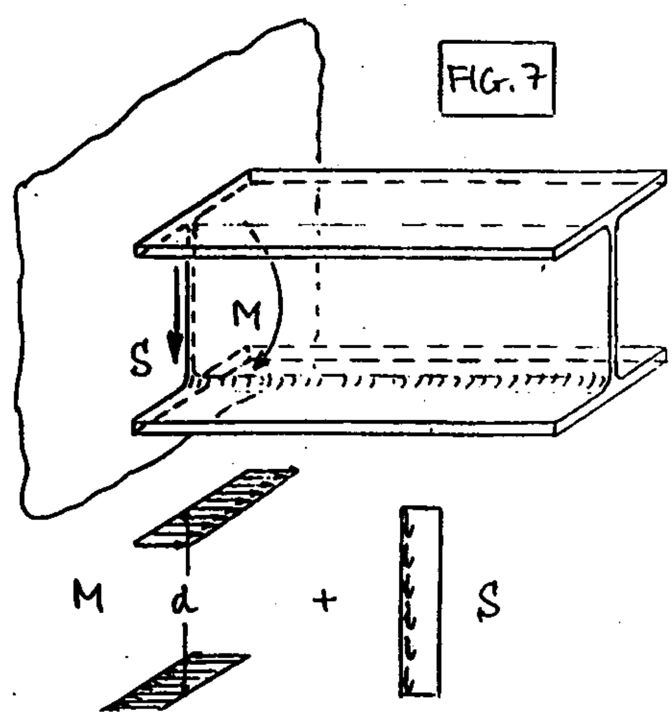


FIG. 8

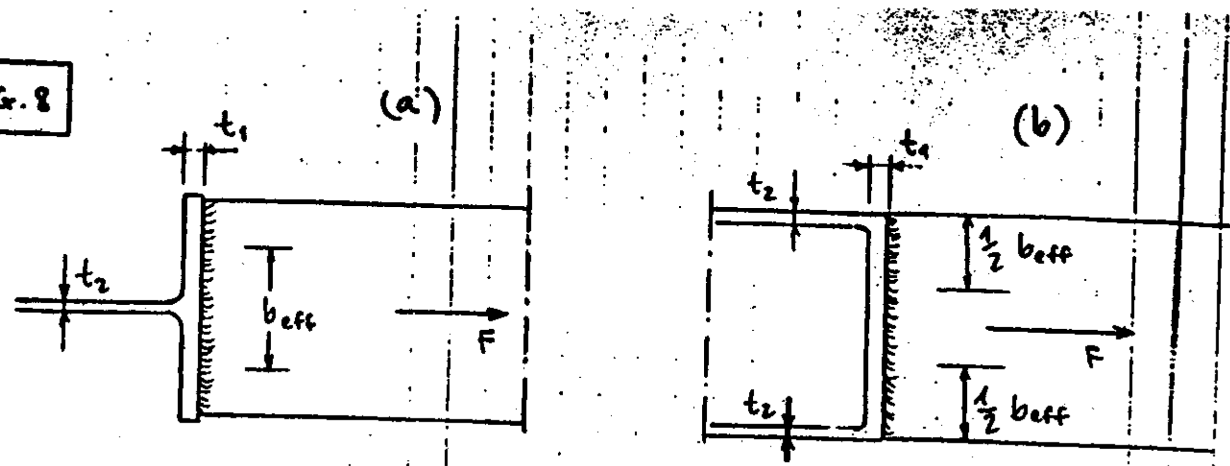
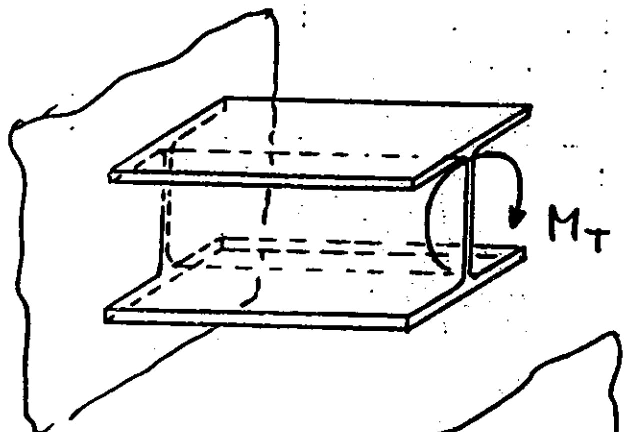
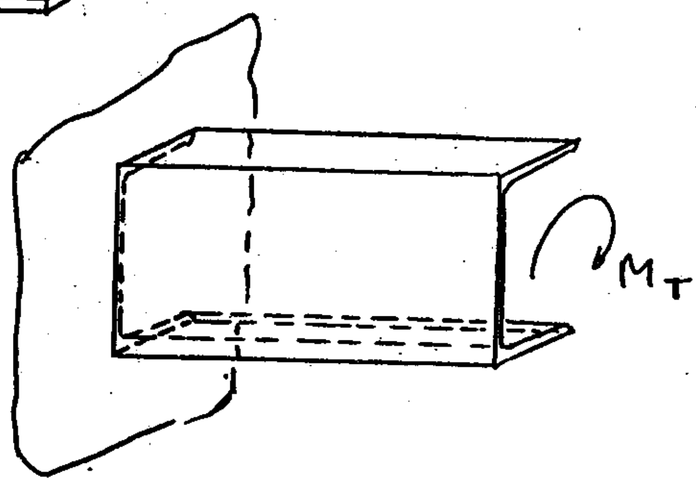


FIG. 9

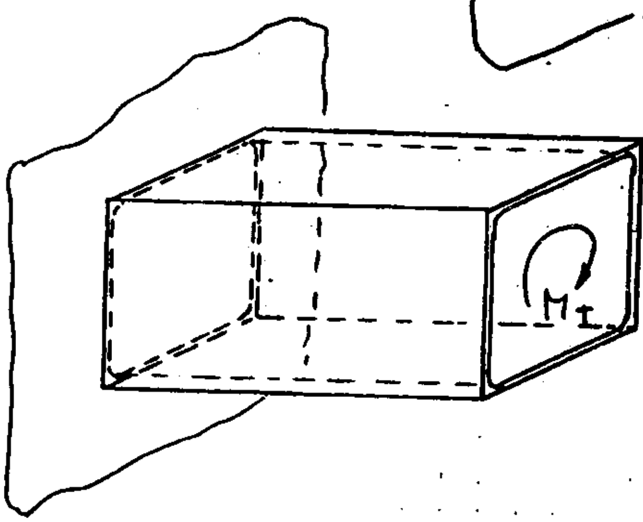
(a)



(b)



(c)



Cases (a) and (b)
St. Venant and/or
warping torsion

Case (c) torsion of
closed sections

9. Appendix 1

Footnotes

1) The reasons for this approach, which includes residual stresses, are given in reference [3.5]. See also the other references of topic 3 in Appendix 2.

2) The formula for the comparison stress was the most controversial question during the discussions of the design rules by commission XV. Starting from the tests of Van der Eb [2.1] and his empirical formula $\sigma_c = \sqrt{\sigma_\perp^2 + 1.8\,(\tau_\perp^2 + \tau_{/\!/}^2)}$ many different types of specimens were tested to find the best approach. Tests show that the strength of fillet welds under combined stresses is roughly represented by an ellipsoid in the $\sigma_\perp, \tau_{/\!/}, \tau_\perp$ - space. However, the shape of the ellipsoid and certain distortions of it depend on many variables, the most important ones being the orientation of the notch to the direction of loading, the type of electrode, the ratio of strength of electrode to base material, the ratio of weld thickness to plate thickness, and whether the loading is in tension or in compression. Basically the difficulty is that one tries to explain complex rupture tests by a formula which is a modified yield criterion. Thus the scatter of test results is quite high and any formula resembling an ellipsoid can be reasonably fitted to the test results (A good survey of all these possibilities is given in reference [2.6]).

Subcommission XV-A has chosen the form $\sigma_c = \beta\sqrt{\sigma_\perp^2 + 3(\tau_{/\!/}^2 + \tau_\perp^2)}$ since it is relatively simple, fits the test results on the safe side and uses the familiar factor 3 with the shear stresses.
The Italian Delegation favors the formula $\sigma_c = \beta\sqrt{\sigma_\perp^2 + 2\tau_{/\!/}^2 + 3\tau_\perp^2}$ with $\beta = 0.8$ for Fe 360 and $\beta = 1.0$ for Fe 510, which gives more conservatice results for fillet welds loaded by $\sigma_\perp$ and $\tau_\perp$.

3) This ratio assures that the adjacent parent material yields before rupture of the welds. Thus a sudden breakdown caused by failure of the welds is avoided and redistribution by plastic deformation is possible.

4) Formula (7.2) neglects the stress peaks at the ends of the welds in the elastic stress distribution, but all rupture tests have shown that it is on the safe side, since the stress peaks are equalized in the loading process by plastic deformation. For thin long welds the deformation capacity at the ends may be exhausted before the middle part of the weld has reached full yield stress, thus causing the connection to fail by a kind of zipper-effect. The different national standards have therefore prescribed a minimum ratio of weld thickness to plate thickness and put limits to the length of the side welds, e.g. max l = 100.a.

5) The factor 0.8 in formula (7.4) was found from the rupture tests of the International Test Series [4.4] as a safe design value in all cases. The actual stress distribution in a lap-joint with side and front fillet welds and the redistribution of stresses under increased loading is so complicated and depends on so many variables (see references [4.6, 6.2, 6.5, 6.7]) that in normal design work it is not reasonable to use a more exact approach for a marginal gain in strength or saving in weld thickness.

6) The quoted values of the factor c_1 were determined from an extensive test series conducted by Stevin Laboratory which included the measurement of the elastic stress distribution [6.6].

10. Appendix 2

Selected list of relevant documents

Topic 1: Calculation rules

[1.1] Calculation of welded joints submitted to static loads IIW Doc. XV-107-60

[1.2] Calculation Formulae for Welded Connections subject to static loads IIS/IIW-139-64 Weld. World Vol. 2, No. 4, 1964, p. 168/97 (IIW Doc. XV-156-63)

[1.3] Schweissanschlüsse torsionsbeanspruchter Träger mit I-, [- und Z-Querschnitten by E.W. Bornscheuer, Schweissen und Schneiden 13(1961), also Appendix II of Doc. IIS/IIW-139-64

[1.4] Directions for the Calculation of Welded Joints for unalloyed Steel with a Tensile Strength up to 50 kgf/mm^2 and Low-alloy Steel with a Tensile Strength up to 62 kgf/mm^2 - Static loads (Engl. Translation of draft Dutch Standard) Doc. XV-A-6-69 see also revised edition XV-A-19-70

[1.5] Design Rules for Welded Connections in Steel- Discussion paper submitted by D. Feder Doc. XV-A-2-71

[1.6] Design Rules for Arc-Welded Connections in Steel - 2nd revision of Doc. XV-A-6-69 worked out by Sc. XV-A Doc. XV-A-4-71

[1.7] Welded Structural Steelwork with Predominantly Static Loading - Design and Structural Details, Translation of German Standard DIN 4100 Doc. XV-A-17-71

[1.8] Calculation of Statically Loaded Fillet Welds - Comparison of Different National and International Rules by Italian Delegation Doc. XV-A-5-73 (ISO/TC44/Sc 2 / secretariat 36/71 E)

[1.9] Calculation of Welded Joints in Unalloyed and Low-alloy Steel up to and including Fe 510 (Fe 52) which are predominantly Statically Loaded - Translation of Dutch Standard NEN 1062-VI Doc. XV-A-9-73 (Doc. XV-A-3-72)

Topic 2: Comparison stress formula

[2.1] Experiments on Fillet Welds by Vander Eb and Vreedenburgh T.N.O. report

[2.2] Tests on Fillet Welds by F.K. Ligtenberg Doc. XV-170-64 (T.N.O. report No. BI-63-33)

[2.3] Tests on Steel Fillet Welds made by Different Filler Metals for the Verification of the Ellipsoid Shape by Italian Delegation Doc. XV-A-5-66

[2.4] Fillet Welds on Steel 52 kg/mm^2 made by different Filler Metals: Tests for the Verification of the λ-Coefficient in the ISO-Formula by U. Guerrera and U. Girardi IIW Doc. XV-227-67

[2.5] The Influence of Parent and Filler Metal on the Value of the λ- and β-Coefficient in

$$\sigma_c = \beta \sqrt{\sigma_\perp^2 + \lambda\left(\tau_\perp^2 + \tau_\parallel^2\right)}$$

for static loaded Fillet Welds in Fe52 by J. Berenbak and R.P. Dommerholt IIW Doc. No. XV-277-69 (Doc. XV-A-18-69, Stevin Lab. Rep. No. 6-69-8)

[2.6] The Simplified Combination Formula Proposed in the new Italian Regulations for the Calculation of Fillet Welds by U. Guerrera, U. Girardi and G. Negri Doc. XV-A-16-71

[2.7] Le Calcul des Cordons d'Angles Rectangles Isocèles Sollicités Statiquement par le Critère de Cisaillement Maximum - 1re Partie, Considérations Théoriques par W. Chapeau, P. Guiaux, J.C. Lambert (CRIF) Doc. XV-A-8-72

[2.8] Considérations au Sujet du Projet de Resolution III de la 4ième Réunion de L'ISO/TC44/Sc2 par W.Chapeau et P. Guiaux Doc. XV-A-3-73

Topic 3: $\sigma_{//}$

[3.1] The influence of a Compressive Longitudinal Stress ($\sigma_{//}$) on the Strength of Fillet Welds - Interim Report Doc. XV-A-8-69 (Stevin Lab. Rep. 6-69-4)

[3.2] The Influence of a Longitudinal Stress ($\sigma_{//}$) on the Strength of Fillet Welds by J.W. van't Hullenaar and J. Strating IIW Doc. XV-274-69 (Doc. XV-A-2-69 Stevin Lab. Rep. 6-69-3)

[3.3] The influence of a Compressive Longitudinal Stress ($\sigma_{//}$) on the Strength of Fillet Welds by L.A.G. Wagenmans IIW Doc. XV-275-69 (Doc. XV-A-16-69, Stevin Lab. Rep. No. 6-69-9-HL 14)

[3.4] Zur Systemumlagerung querbelasteter, geschweisster Biegeträger von F.W. Bornscheuer und G. Werner Doc. XV-A-9-70, Doc. XV-A-12-71 (reprint) (Paper for IV. Schweisstechn. Hochschulkolloquium, Essen, April 1970)

[3.5] The Influence of a Longitudinal Stress on the Strength of Fillet Welds - Final Report by J. Strating, A.A. van Douwen, J.W. van't Hullenaar and L.A.G. Wagenmans IIW Doc. XV-315-71 (Doc. XV-A-5-71, Stevin Lab. Rep. No. 6-71-7)

[3.6] Statische und dynamische Untersuchungen zur Feststellu des Einflusses der Längsspannungen $\sigma_{//}$ auf das Tragver halten geschweisster Biegeträger
by F.W. Bornscheuer and G. Werner
IIW Doc. XV-A-3-74 (Reprint from Schweissen und Schneiden 26(1974), p. 37/40

Topic 4: International Test Series and similar tests

[4.1] Contribution to the International Test Series by I. Gallik IIW Doc. XV-233-67

[4.2] International Test Series - Final Report, March 68, revised edition of Doc. XV-225-67 Doc. XV-a-4-68 IBBC: BI-68-25

[4.3] Tests on welded connections with long or thick fillet welds by F.W. Bornscheuer and D. Feder

[4.4] International Test Series - Final Report by F.K. Ligtenberg IIW Dox. XV-242-68 (IIW Doc. XV-225-67)

[4.5] Resultats d'Essais sur des Assemblages avec des Soudures d'Angle by V. Popescu and C. Dalban IIW Doc. XV-258-68 (Doc. XV-A-5-68)

[4.6] Some Experimental Studies on Fillet Weld Lap Joints by A. Okukawa, K. Horikawa and T. Okumura Doc. XV-A-16-70

Topic 5: Strength of welds

[5.1] Deformation and Strength of End Fillet Welds by Takeo Naka and Ben Kato IIW Doc. XV-A-9-65

[5.2] Einfluss der Nahtdicke auf die statische Festigkeit von Flankenkehlnähten by D. Feder, Schweissen und Schneiden 19(1967), p.299/305 distributed for information in Sc. XV-A, no number

[5.3] Utilization of the Penetration for Fillet Welds by Swedish Institute for Production Engineering Research IIW Doc. XV-268-69 (Doc. XV-A-20-69)

[5.4] Preliminary Report on the 'Strength of Fillet Welds made by Automatic or Semi-automatic Welding Processes by J. Strating Doc. XV-A-20-70

[5.5] The Strength of Fillet Welds made by Automatic and Semi-Automatic Welding Processes by J. Strating IIW Doc. XV-316-71 (Doc. XV-A-6-71 Stevin Lab. Rep. No. 6-71-6-HL 13)

[5.6] Static Strength of Welded Connections with Side Fillet Welds - Design of a New Type of Specimen and Comparative Investigations by G. Werner Doc. XV-A-11-71

[5.7] A new Type of Specimen for Testing Side Fillet Welds by G. Werner IIW Doc. XV-327-72

[5.8] Le Calcul des Cordons d'Angles isocèles Sollicités Statiquement par le Critère de Cisaillement Maximum - 4ième partie: Vérification expérimentale dans le cas d'assemblages en acier trempé revenu par W. Chanaeu et P. Guiaux Doc. XV-A-4-73

Topic 6: Certain types of connections

[6.1] Welding Seams in Beam-Column Connections without the Use of Stiffening Plates by L.W.A. v. d. Elzen IIW Doc. XV-213-66 (Stevin Lab. Rep. No. 6-66-2)

[6.2] Stresses in Fillet-welded Lap-joints by D. Feder IIW Doc. XV-A-8-67

[6.3] Effective weld length of Beam to Column Connections with and without stiffeners by P.J. de Geeter IIW Doc. XV-244-68 (Stevin Lab. Rep. No. 6-68-1) (IIW Doc. XV-A-3-68)

[6.4] Effective Weld Length of Beam to Column Connections without Stiffeners - Interim Report Suppl. Report to IIW Doc. XV-213-66 and IIW Doc. XV-244-68 Doc. XV-A-9-69 Stevin Lab. Rep. No. 6-69-5

[6.5] The Strength of Fillet Welded Joints by B. Kato and K. Morita IIW Doc. XV-267-69 (Doc. XV-3-69

[6.6] The effective Weld Length of Beam to Column Connections without Stiffening Plates - Final Report by A. Rolloos IIW Doc. XV-276-69 (Doc. XV-A-17-69 Stevin Lab. Rep. No. 6-69-7-HL 12)

[6.7] Elastic and Plastic Stress Distribution on Base Plates in the Neighbourhood of Fillet Welds in Lap Joints by K. Horikawa and T. Okumura IIW Doc. XV-290-70 (Trans. Jap. Weld. Soc. Vol. 1 No. 1, 1970) Doc. XV-A-6-70

[6.8] Deformation and Stress due to Welding of Typical Welded Joints in a Steel Building by F. Matsushita, H. Nakayama, M. Matsumoto and M. Inagaki IIW Doc. XV-292-70 (Doc. XV-A-14-70)

[6.9] The Maximum Strength of Beam to Column Connections without Stiffening Plates by B. Kato, K. Morita, K. Hashimoto IIW Doc. XV-311-71

Topic 7: Influence of material

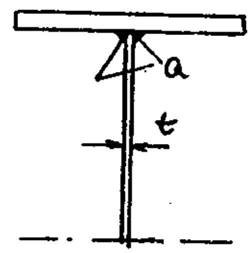
[7.1] Influence of the Steel quality on the Strength of Steel Constructions IIW Doc. XV-278-69 (Doc. XV-A-19-69 Stevin Lab. Rep. No. 6-69-1)

[7.2] Influence of the Steel quality on the Strength of Steel Constructions Doc. XV-A-17-70(Stevin Lab. Rep. 6-70-15)

11. Appendix 3

E x a m p l e s

> **Note:** For all cases load factor design is assumed to eliminate the question of safety factors.
>
> Thus $\sigma_c = \sigma_e = 240\ \text{N/mm}^2$ for Fe 360* $\}$
> $= 360$* " for Fe 510 $\}$ *buckling confusion
>
> Slide rule accuracy is used throughout.



Example 1: Weld between flange and web of an I-section

loading of welds: shear flow $S = 3000$ N/mm

$\longrightarrow$ stresses in welds: $\tau_{//}$ only

from formula (6.1) $0.7\sqrt{3\,\tau_{//}^2} = \sigma_c$ $\qquad$ $0.85\sqrt{3\,\tau_{//}^2} = \sigma_c$

$\longrightarrow$ allowable $\tau_{//} = 198$ N/mm² for Fe 360 $\qquad$ $\tau_{//} = 245$ for Fe 510

This is allowable stress for welded joint with shear in ‖ only.

> The above values can be taken directly as the allowable stresses for fillet welds loaded by shear flow only

required weld thickness in this example for Fe 360

$$a = S/2\,\tau_{//} = 3000/2\cdot 198 \sim 8\ \text{mm}$$

Comparing the permissible shear stresses in the parent material and the welds one gets the following direct relation between web thickness and weld thickness:

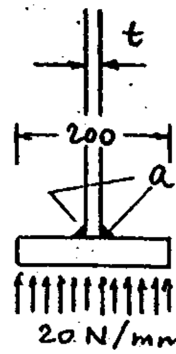
$$a = 0.35\cdot t\cdot\frac{\tau_{actual}}{\tau_{allowable}} \qquad a = 0.4\cdot t\cdot\frac{\tau_{actual}}{\tau_{all.}}$$

for Fe 360 $\qquad$ for Fe 510

Since in most cases the web will be designed for the full allowable stress, a safe formula in all cases will be

$$a = 0.4\cdot t$$

Example 2: Weld between web or stiffener and base plate



assume pass fit up & all force applied via welds.

loading of welds: linear pressure (force / unit length) $q = 4000$ N/mm

⟶ stresses in welds: $\sigma_{\perp} = \tau_{\perp} = p/\sqrt{2}$

see p. 11 fig 4. with $p = q/2\,a\,(\ell)$ $\ell = 1$

from formula (6.1)

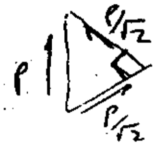
$$0.7\sqrt{p^2/2 + 3\cdot p^2/2} = \sigma_c$$

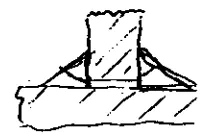
really want τ + σ in this eq.

$$= 0.7\sqrt{(\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2))}$$

But, no $\tau_{\parallel}$

$$= 0.7\,p\sqrt{4/2} \;\ldots\ldots$$





$0.7\cdot\sqrt{2}\cdot p = \sigma_c$	$0.85\cdot\sqrt{2}\cdot p = \sigma_c$
$p \approx \sigma_c$	$p = 0.83\ \sigma_c$
$p = 240$ N/mm²	$p = 300$
for Fe 360	for Fe 510

The above values can be taken directly as the allowable stresses for fillet welds loaded in the manner indicated (compression or tension)

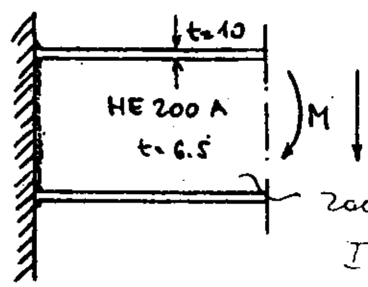
required weld thickness in this example for Fe 360

$$a = q/2p = 4000/2\cdot 240 \approx 8.5 \text{ mm}$$

Comparing the permissible stresses in the parent material and in the welds for this loading case one gets the following relation between web thickness and weld thickness:

$a = 0.5\cdot t$	$a \approx 0.6\cdot t$
for Fe 360	for Fe 510

Example 3: Beam-column connection **with** stiffeners

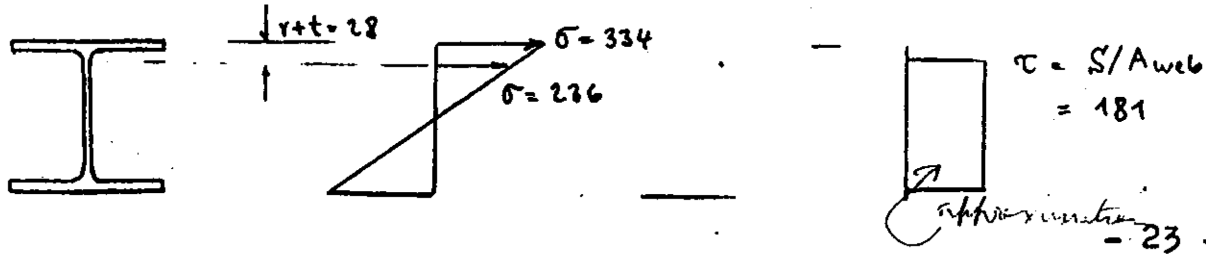


200×180×20. approx
$I = 38\cdot 10^6$ mm⁴

assumed: steel Fe 510 $S = 200$ kN
$M = 130$ m kN

(a) Elastic analysis

stresses in parent material of girder [N/mm²]



$$a_{flange} = (334 \cdot 10)/(2 \cdot 300) = 5.6 \text{ mm}$$

see example 2

or $$a_{flange} = 0.6 \cdot t = 6 \text{ mm}$$

To calculate a_{web} formula (6.1) has to be used again, because of the presence of $\tau_\perp$, $\sigma_\perp$ and $\tau_{//}$.

With $\tau_\perp = \sigma_\perp = \frac{\sigma}{\sqrt{2}} \cdot \frac{6.5}{2\,a}$ and $\tau_{//} = \tau\,\frac{6.5}{2\,a}$

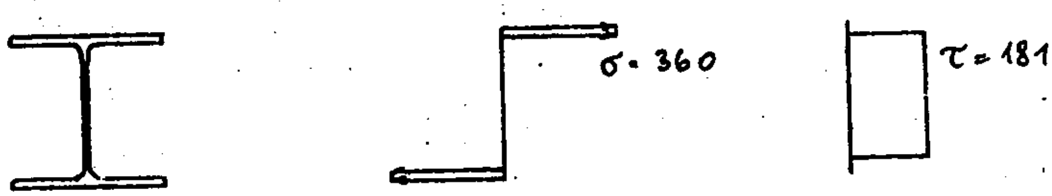
$$0.85 \cdot \frac{6.5}{2\,a} \sqrt{2\,\sigma^2 + 3\,\tau^2} = 360$$

$$0.85 \cdot \frac{6.5}{2\,a} \sqrt{2 \cdot 236^2 + 3 \cdot 181^2} = 360$$

$$\rightsquigarrow a_{web} = 3.5 \text{ mm}$$

(b) Plastic analysis

stresses in parent material of girder $[N/mm^2]$



$$a_{flange} = (360 \cdot 10)/(2 \cdot 300) = 6.0 \text{ mm}$$

$$a_{web} = (181 \cdot 6.5)/(2 \cdot 245) = 2.4 \text{ mm}$$

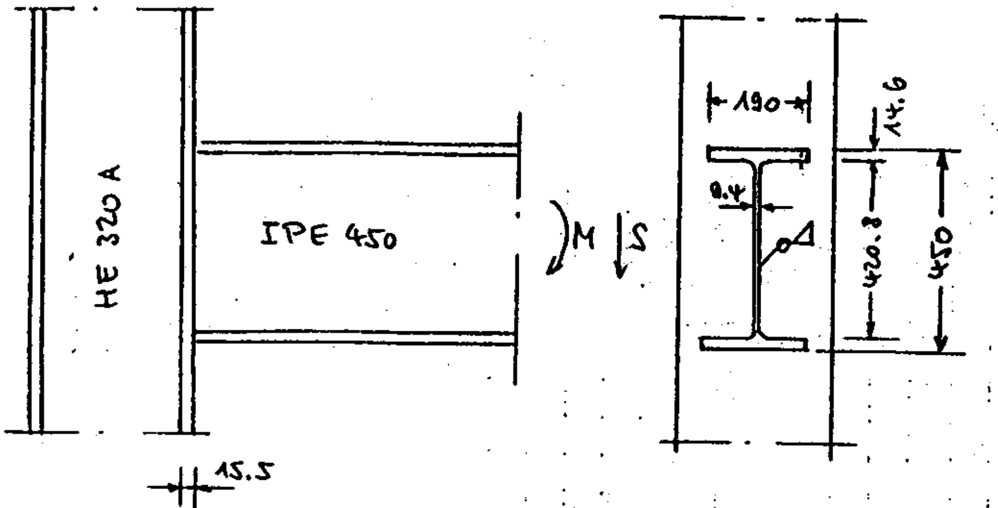
see example 1

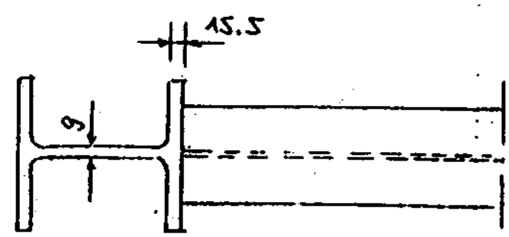
check for deformation capacity:

$$a_{flange} = 0.7 \cdot \frac{360 \cdot 10}{2 \cdot 300} = 4.2 < 6.0 \text{ mm}$$

$$a_{web} = 0.7 \cdot \frac{360 \cdot 6.5}{2 \cdot 300} = 2.7 > 2.4 \text{ mm}$$

Example 4: Beam-column connection without stiffeners





assumed values:
steel Fe 360
$M = 172$ m kN
$S = 500$ kN

From formula (7.7) and the relevant table we get

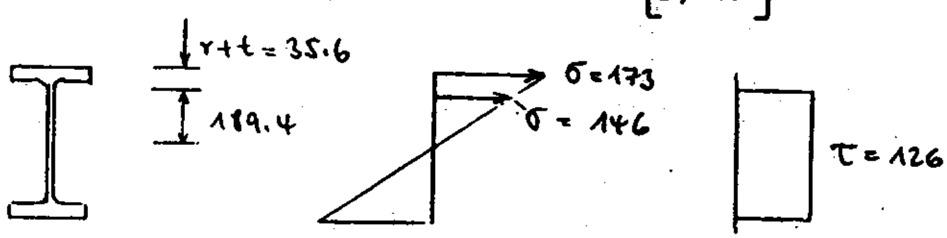
$$b_{eff} = 7 \cdot 15.5 + 2 \cdot 9 = 126 \text{ mm}$$

For the sake of simplicity all normal stresses in the girder are assumed to increase by the factor b/b_{eff} .

joint factor.

(a) Elastic analysis

stresses in the parent material $[N/mm^2]$



minimum σ for which flange-welds have to be calculated

$$\min \sigma = 0.7 \cdot \sigma_e = 0.7 \cdot 240 = 168 < 173$$

$$a_{flange} = \frac{173 \cdot 14.6}{2 \cdot 240} = 5.3 \text{ mm}$$

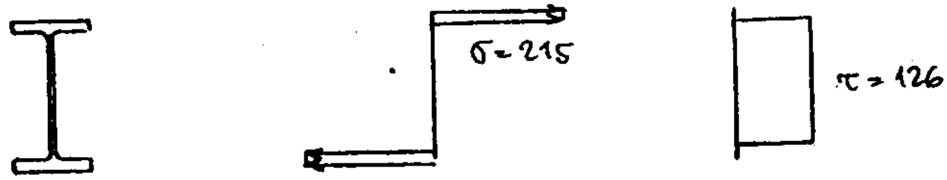
see example 2

$$a_{web} = 0.7 \frac{9.4}{2 \cdot 240} \sqrt{2 \cdot 146^2 + 3 \cdot 126^2} = 4.1 \text{ mm}$$

(analogous to example 3 (a))

(b) Plastic analysis

stresses in the parent material N/mm^2



min σ for a_{flange} = 168 < 215

$$a_{flange} = \frac{215 \cdot 14.6}{2 \cdot 240} = 6.5 \text{ mm}$$

$$a_{web} = \frac{126 \cdot 9.4}{2 \cdot 198} = 3.0 \text{ mm}$$

see example 1

check for deformation capacity:

$$a_{fl.} = 0.7 \frac{240 \cdot 14.6}{2 \cdot 240} = 5.1$$

$$a_{web} = 0.7 \frac{240 \cdot 9.4}{2 \cdot 240} = 3.3$$

Example 5: Connection of an angle to a gusset plate

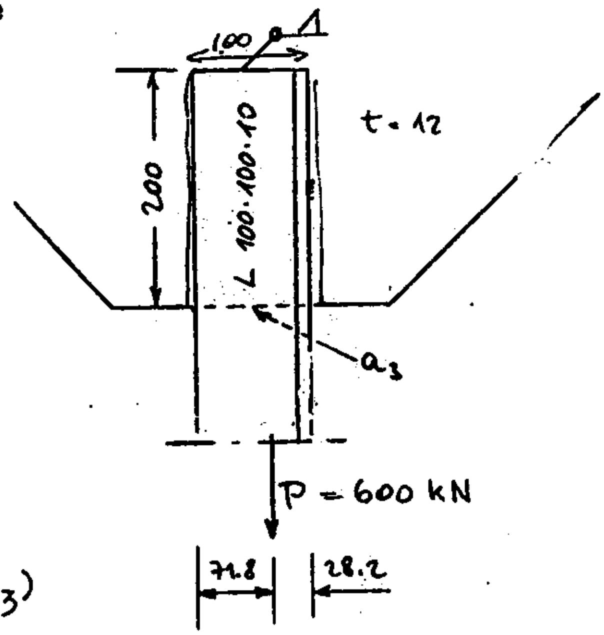
steel Fe 510

1st assumption: all welds equal thickness

according formula (7.3)

$$P = (a \cdot 400 \cdot 245 + a \cdot 300 \cdot 200)$$
$$= a \cdot 158000$$
$$a = 600/158 = 3.8 \text{ mm} \quad (\text{with } a_3)$$

$$P = (a \cdot 400 \cdot 245 + a \cdot 300 \cdot 100)$$
$$= a \cdot 128\,000$$
$$a = 600/128 = 4.7 \text{ mm} \quad (\text{without } a_3)$$



To take the moment caused by the eccentricity of the centroidal line of the angle we assume a couple of forces in the side welds:

$$P_M = 600 \cdot 21.8 / 100 = 131 \text{ kN} = a \cdot 200 \cdot 245$$
$$a = 131/49 = 2.7 \text{ mm}$$

i.e. the weld at the free leg of the angle has to be increased

to 3.8 + 2.7 = 6.5 mm thickness

(4.7.+ 2.7 = 7.4)

whilst the other side weld could be reduced in thickness to the minimum possible value of a = 3 or 3.5 mm.

12. List of symbols

a thickness of fillet weld (see Fig. 2)

l length of fillet weld (see paragraph 5.3)

A surface of throat section of fillet weld (see par. 5.3)

t plate thickness

$\left.\begin{array}{l}\sigma_{/\!/}\\ \sigma_{\perp}\\ \tau_{/\!/}\\ \tau_{\perp}\end{array}\right\}$ stresses in fillet weld (see par. 6.2 and Fig. 4)

σ_c comparison stress (see formulae (6.1) and(6.2))

σ_e yield stress of parent material ("e" for limit of elasticity)

$\overline{\sigma_t}$ allowable tensile stress in parent material

$\overline{\sigma_p}$ stress in parent material

$\overline{\sigma_w}$ allowable stress in fillet weld for a certain kind of loading

ε_{10} elongation measured over a length equal $\sqrt{10}$ times the cross-sectional area of the standard tensile test piece

p - resultant of stress components acting on weld throat. (Fig 4)