The Significance of Weld Discontinuities Regarding Plastic Failure

A. M. GRESNIGT

This paper was presented during the IIW Annual Meeting 1985 in Strasbourg on a joint seminar on the application of fitness-for-purpose criteria in welding. It was organized by commissions V (testing, measurement and control of welds), X (residual stresses and stress relieving, brittle fracture), XIII (fatigue of welded components and structures) and VX (fundamentals of design and fabrication for welding). At the end of the meeting it was decided to prepare a provisional standard document for guidance on the significance of discontinuities on a fitness-for-purpose basis in collaboration with these commissions.

The present paper gives some considerations and motives which have led to the "Code of practice for requirements applicable to the shape and homogeneity of welds in unalloyed and low alloy steel, with a specified yield point up to and including 355 N/mm² " (51 ksi). This code has been published by the IIW, Commission XV.[3] Furthermore, the Eurocode 3 drafting panel has adopted this code.[11]

1. INTRODUCTION

The answer to whether weld discontinuities affect the correct functioning of welded joints, depends on the demands made on those joints, the extent of the weld discontinuities and the mechanical properties of the weld metal and the base metal.

In Sect. 2, a summary is given of the demands which are made on welded structures. Section 3 demonstrates briefly how the "Design rules for arc-welded connections in steel submitted to static loads"[1] meet these demands. The influence of weld discontinuities will then be discussed. Finally, conclusions and recommendations are made.

2. DEMANDS MADE ON WELDED STRUCTURES

These demands may simply be explained by considering the structure in Fig. 1 connections in, for instance, a multi-story building.
The demands are:

a. *Strength*. The connections must be able to carry the loads.
b. *Stiffness*. The connections must be stiff enough to prevent unacceptable deformations.
c. *Deformation capacity*. In statically indeterminate structures, actual stresses can be quite different from calculated stresses. This may be caused by a number

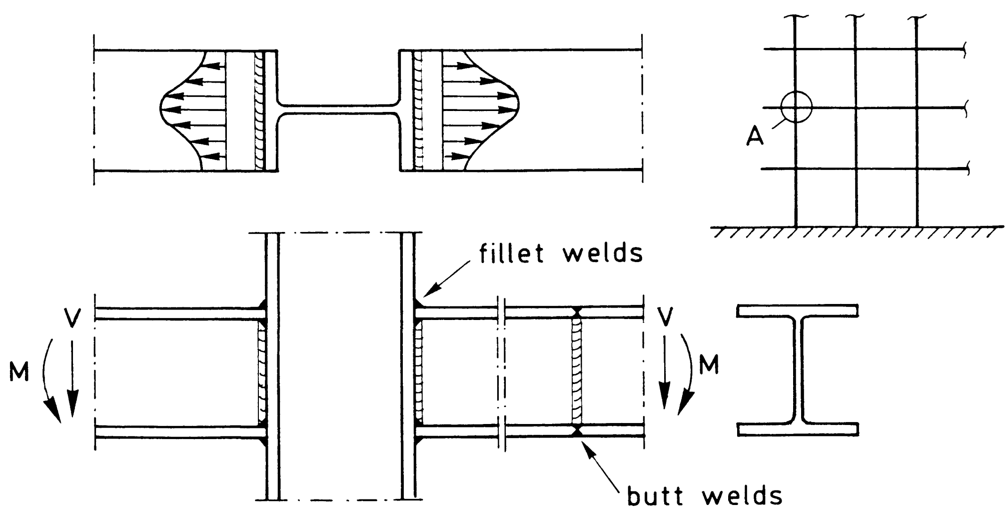


Fig. 1. Welded connections, detail A in multi-story building. Left hand beam is statically determinate; right hand beam is statically indeterminate.

A. M. Gresnigt is with the TNO Institute for Building Materials and Structures, Delft, The Netherlands.

of factors, generally not included in calculations, for instance:

- Differences between the actual dimensions of the structural parts and the theoretical dimensions
- Temperature differences in the structure
- Unequal settlements

The maximum stresses are limited because of yielding. If the ultimate strength of the weld is lower than the yield strength of the adjacent members, then all deformations will occur in the welds. Because of the dimensions of the welds, these deformations are generally not sufficient to meet the required deformations. Sufficient deformation capacity can be obtained when the welds are stronger than the yield strength of the weakest of the adjacent members

$$F^r_{uw} > F^r_{ym} \quad (1)$$

F^r_{uw} = required ultimate strength of the weld

F^r_{ym} = real yield strength of the adjacent member.

This demand is also applicable to statically determinate structures when stress concentrations occur due to differences in the stiffness of the various parts in the joint. See the stress distribution near the fillet welds in Fig. 1.

The conclusion from the above is that in statically indeterminate structures, and in many cases statically determinate structures, a sound performance only can be obtained if the real ultimate strength of the weld is larger than the real yield strength of the weakest of the adjacent members.

Although deformation capacity of the welds themselves is often not required for the "overall" structural behavior, it is particularly required to account for local yielding in the welds. The lap joint of Fig. 2 is an example.

Because of the possibility for local yielding it is allowed to neglect local stress concentrations, and to add the full strength of the side fillet welds and the end fillet weld for the determination of the strength of the joint.

The demands made on local yielding depend on the thickness of the plates. In thicknesses up to 0.4–0.6 in. (10–15 mm), the demands are less severe than for greater thicknesses. If the thickness is greater than 1.2–1.6 in. (30–40 mm), this point needs real attention. Of course, these figures are approximate because not only the thickness is important but also the form of the connection, the length of the weld etc.

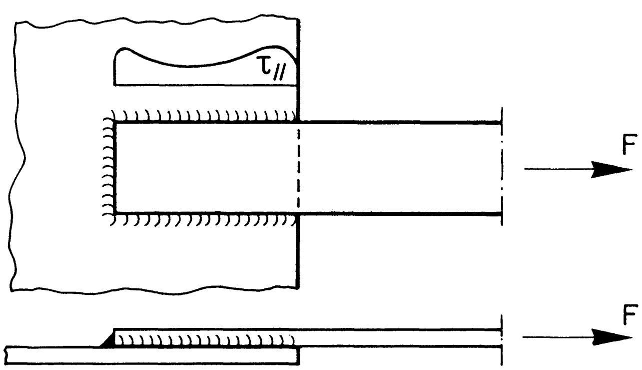


Fig. 2. Lap joint

3. DESIGN RULES

In 1976, Commission XV has published their "Design Rules for Arc-welded Connections in Steel Submitted to Static Loads."[1] These design rules are based on extensive, mainly experimental programs. See for example the report on the "International Test series."[2] In these test series, a large number were carried out in more than 10 countries. Each country applied the steel qualities and welding processes locally available.

Based on theoretical considerations and on these test results, the design formulas were established. When applying the design formulas to fillet welds designed to carry the yield strength of the connected member, it was found that for Fe 360 steel (minimum yield strength for thicknesses up to 0.6 in. (16 mm): 34 ksi (235 N/mm^2); minimum tensile strength 52 ksi (360 N/mm^2) , the fillet welds are able to carry at least 140% of the *real* yield strength of the connected member:

$$F_{uw} \geq 1.4\ F^r_{ym} \quad (2)$$

F_{uw} = ultimate strength of the fillet welds

F^r_{ym} = real yield strength of the member

Together with the demands in Formula 1 it can be concluded that if due to weld defects the real strength of the weld is lowered by 30% ($0.3 \times 1.4 = 0.4$), the weld is still fit for its purpose.

For butt welds, the same is applicable. In case of an incomplete penetration of 30%, the butt weld can still carry the real yield strength of the adjacent plates. For Fe 510, these figures may be somewhat smaller.

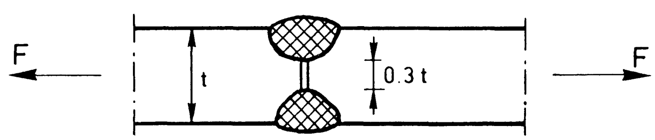


Fig. 3. Butt weld with incomplete penetration

Of course, the above holds if the welds are designed within the range of application of the design rules[1] and the conditions about the correct choice of base and filler materials and yielding procedure are fulfilled. A copy of Sects. 2 and 3 of the design rules[1] is given below.

2. Range of application

These recommendations are intended for the calculation of statically loaded welded connections in carbon and low alloy steels having a minimum tensile strength less than 87 ksi (600 N/mm^2), a ratio of yield to ultimate strength ≤ 0.8, and a minimum elongation $\epsilon_{10} \geq$

12%. The welds are assumed to be made by arc welding (i.e. covered electrode, gas shielded or submerged arc welding).

3. Workmanship and techniques

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

3.1. The design must take into the particular characteristics of the welded connections.

3.2. The correct choice of base and filler materials must be made in respect to:

- 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 must be carried out in accordance with drawings and specifications which indicate where necessary,

- the types and dimensions of the welds
- limiting conditions for welding
- the welding process
- the welding sequence
- the type of filler materials
- the extent and method of inspection
- the acceptability of defects
- special requirements to which the connection must conform.

3.4. A sound welded joint must be assured by adequate supervision of:

- the preparation and assembly for welding
- the welding procedure
- welders with the necessary skills for the task.

From these two sections it may be concluded the conditions for the choice of base and filler material and on the welding process, etc., are not very specific. Usually, the designer of the steel structure is not an expert in this field, and because most steel structures on shore are often designed by rather small design offices who usually do not have welding engineers, the choice of the filler material and welding process etc. has to be made by the fabricator. Small fabricating yards, where a lot of steel structures are fabricated, do not often have a welding engineer either.

Of course, the above is not applicable to structures such as pressure vessels, pipelines, offshore platforms etc. For these structures, special codes exist and it may be expected that these structures are designed and fabricated by design offices and fabricators who have the right experts.

With respect to these observations, Commission XV has agreed upon a "Code of practice for requirements applicable to the shape and homogeneity of welds in unalloyed and low alloy steel, with a specified yield point up to and including 51 ksi (355 N/mm^2), subject to predominantly static loading."[3] The preface of this document is:

PREFACE

This code of practice was initially drafted by a Working Group from within the standardisation committee "Weld calculations" of the Netherlands Standards Institution (NNI). It was published in 1977.

Since its publication many manufacturers and clients have made and continue to make the document a part of their contract documents.

An English translation of the Dutch code of practice was presented and discussed by Commission XV of the IIW (doc. XV-406-77). In the discussion it appeared that there are different views with respect to the obligations and liabilities of the interested parties. Sections 3 and 4 deal with these obligations and liabilities. In most European countries the provisions of these sections are acceptable since they follow common practices established between manufacturers, clients and inspectors. In the U.S. however, this is not the case.

From discussions in Commission XV it was apparent that it was impossible to arrive at a suitable wording for the provisions having potential legal implications that would be agreeable to all. It was decided to keep the Sects. 3 and 4 in their original format and to state that users, who intend to use the document as a part of their contract, may feel free to amend those particular sections in a way they deem necessary.

There were only minor technical changes proposed by Commission XV. Where accepted these changes have been incorporated in this revised edition of the document.

The code of practice is intended for rather simple and predominantly statically loaded steel structures. Clause 1.1 gives a more specific definition.

Furthermore it is intended for use in conjunction with design rules laid down in doc. IIS/IIW-504-76: "Design rules for arc-welded connections in steel submitted to static loads" (*Welding in the World*, Vol. 14, No. 5/6, 1976).

It should also be recognized that the sound service performance of a welded structure does not solely depend on the structure meeting the strength requirements (design) and workmanship criteria (fabrication), but amongst others additionally on

- A judicious selection of the base metal and
- The use of optimum parameters in the established welding procedure including due consideration of heat treatment whether related to applicable restraint conditions and/or to prevention of potential hydrogen embrittlement. In the case of the former more restrictive allowances for discontinuities may be an alternate or complementary measure.

The acceptability criteria for the pertinent types of discontinuities have been based on the collective experience and judgment of Commission XV. It is the expectation of the Commission that this code of practice will serve as a guideline and provide effective assistance to manufacturers, clients and inspectors.

By publishing this document, guidance was given on the questions about the acceptable weld defects. However, until now, the IIW has not yet published guidelines for the "judicious selection of the base metal," and the "use of optimum parameters in the established welding procedure . . ." which can be used in the mentioned design offices and welding shops that do not have a qualified welding specialist.

A correct choice of the quality of the base metal, the filler metal and the welding procedure is necessary to avoid brittle weldments, which, in combination with some weld discontinuities may cause brittle failure.

As seen in the beginning of this section, a weakening up to 30% in the welds may still give a fit-for-purpose performance of the structure. This means when ductile weldments are obtained, even in the case of a large number of weld defects (discontinuities), the structure may be fit for its purpose. It is pointed out this should not be considered as an invitation to accept such large weld defects. It gives only an idea of the reserves.

4. EXPLANATIONS TO "WORKMANSHIP TOLERANCES AND PERMISSIBLE WELD DISCONTINUITIES—STATIC LOADING"[3]

The influence of the defect size on the ultimate strength of a weld is indicated in Fig. 4. This figure is reproduced from a paper by Prof. Soete and Dr. Denys from Gent University.[4]

The critical defect length a_{cr} follows from the requirement that it must be possible that σ_n is at least σ_y. The line DAFBCW gives an indication of the influence of planar defects. The position of this line depends mainly on

- The ductility of the metal in the cross section where the defect lies
- The position and form of the defect(s)

Gent University has developed methods to determine the correct line for particular situations. In Great Britain, the Crack Tip Opening Displacement Method has been developed (CTOD).[5] With this method, it is possible to establish a quantitative relationship between the position of the defect, its form and size, the ductility of the metal near the defect and the deformation capacity of the plate with the weld containing the defect.

In case of non-planar defects, such as porosity and slag inclusions, in Fig. 4 a line more to the right is applicable, for instance DEW.

In past decades, a large number of tests have been done on welds with all kinds of defects. See Ref. 6 with several hundred references. In 1983, Ben Kato, et al, presented a paper in Commission XV of the IIW on the influence of lack of penetration.[7] In their paper, they reported 57 tests in which the area of the lack of penetration was between 0 and

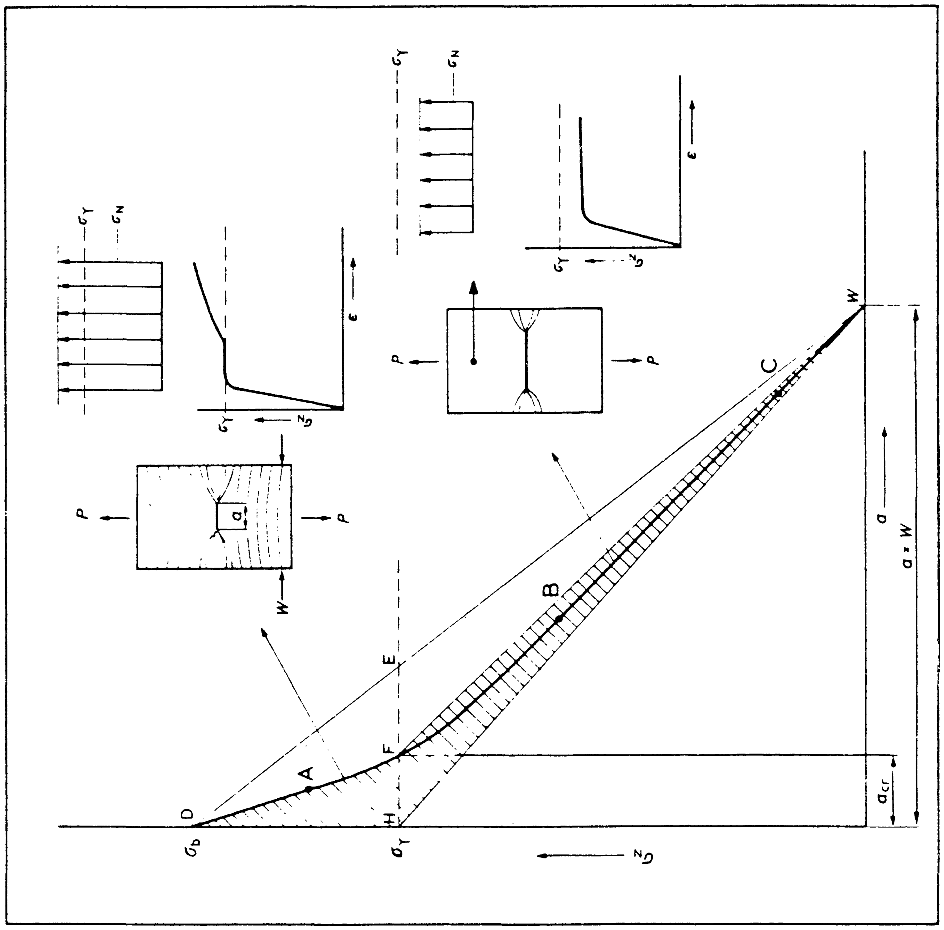
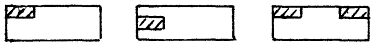


Fig. 4. Influence of defect size on ultimate strength of weld. Symbols are given below.

a = Defect length
a_{cr} = Critical defect length
w = Length of weld
P = Force
σ_y = Yield strength of base metal
σ_b = Ultimate strength of metal in weld
σ_n = Applied stress

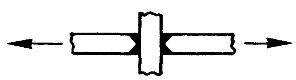
12% of the area of the cross section of the test piece. Further variables were:

- Position of defects: embedded and non-embedded defects



- Temperature: 20°C and −20°C
- Grade of steel : SM 50 ($\sigma_y \simeq$ 52–54 ksi; $\sigma_u \simeq$ 77–78 ksi)
  and SM 58 ($\sigma_y \simeq$ 78–86 ksi; $\sigma_u \simeq$ 91–94 ksi)
- Plate thickness : 0.75 and 1.26 in. (19 mm and 32 mm)
- Welding process: manual arc welding and CO_2 shielded semi-automatic arc welding
- Loading : monotonic and incremental cyclic reversal loading
- Type of joint : butt

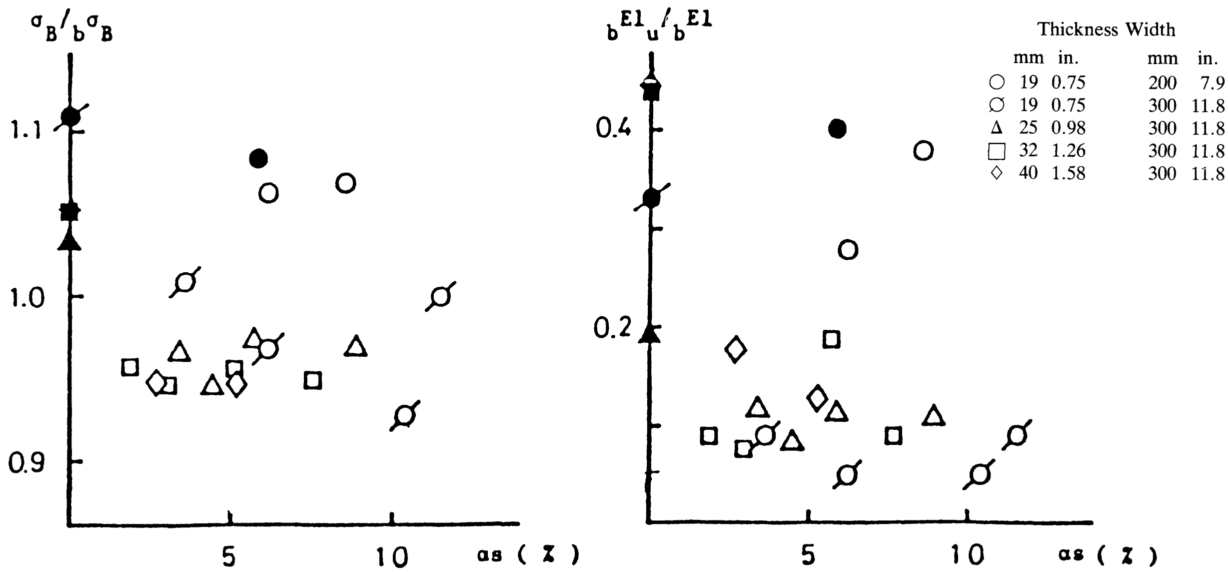
  and cross



From Fig. 5-a it appears the ultimate strength of the specimen was at least 93% of that of the base metal. Since the ratio between the yield strength and ultimate strength of the base metal was lower than this value, it may be concluded that in all tests, the ultimate strength of the specimen was greater than the yield strength of the base metal. Therefore, general yield was obtained. The lowest general yield percentage was 1.4%. Because the ratio ${}_bEl_u/{}_bEl$ is (far) less than 1.0, it may be concluded the major yielding occurred in the base metal.

In the IIW-XV Code of practice,[3] two sets of requirements have been drawn up for evaluation of welds, namely, one set for the *normal quality control level* (the normally required level of quality) and a second set of less stringent requirements for the *minimum quality control level* (the minimum level of quality to be attained).

The requirements for the normal quality control level have been based on what, from the workmanship point of view, can be considered practically achievable. A reasonably skilled welder should be able to work within these requirements. On the other hand, the minimum quality control level is based mainly on strength considerations. The aim is to conform at least to the normal quality control level. If the quality of the welding as actually executed is found to be between the normal quality control level and



(a) Strength

(b) Deformability

Fig. 5. Relationships between αs and strength and deformability. Darkened circles indicate fracture in base metal of test specimen. Symbols are given below.

αs = Lack of penetration in %
σ_B = Ultimate strength of specimen
${}_b\sigma_B$ = Ultimate strength of base metal
${}_bEl_u$ = Ultimate strain in specimen
${}_bEl$ = Ultimate strain in base metal (measured in test piece without welds)

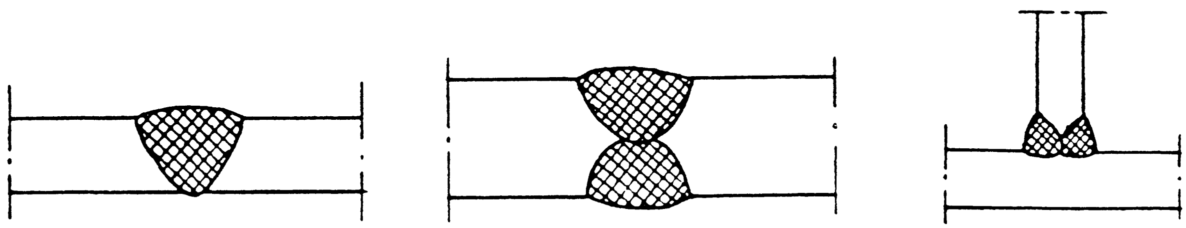
the minimum quality control level, measures should be taken to improve the quality of the welding work still to be executed. The requirements for incomplete penetration in butt welds are:

2.2.1 Normal quality control level	2.2.2 Minimum quality control level

e. Incomplete penetration

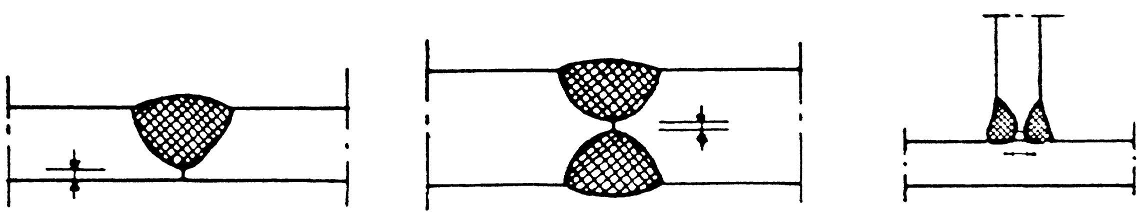
e1 If complete penetration is required for a groove weld, this should be clearly stated and shown on the drawing. For such welds the maximum permissible amount of incomplete penetration is as follows:

Normal quality control level	Minimum quality control level
Within any 6 in. (150 mm) of the weld a total length of incomplete penetration of 0.8 in. (20 mm).	Within any 6 in. (150 mm) length of the weld a total length of incomplete penetration of 1.6 in. (40 mm).



e2 For groove welds, welded from one or from both sides, for which complete penetration is not clearly indicated on the drawings the maximum permissible amount of incomplete penetration is as follows:

Normal quality control level	Minimum quality control level
Depths of incomplete penetration of 5% of the thickness of material or the theoretical throat, with a maximum of 0.08 in. (2 mm)*	Depths of incomplete penetration of 10% of the thickness of material or the theoretical throat, with a maximum of 0.12 in. (3 mm)*



f. Combination of length of slag inclusions and incomplete penetration

Normal quality control level	Minimum quality control level
The sum of the measured identifiable lengths of slag inclusions or of incomplete penetration or the combined length of both must not be more than 4 in. (100 mm) of any 6 in. (150 mm) of the length of weld considered.	The sum of the measured identifiable lengths of slag inclusions or of incomplete penetration or the combined length of both must not be more than 6 in. (150 mm) of any 6 in. (150 mm) of the length of weld considered.

*For the allowable length, see the clause: f. Combination of length of slag inclusions and incomplete penetration.

From these requirements, it can be concluded that for the normal quality control level 5% of 20/150 = 0.7% or 5% of 100/150 = 3.3% of incomplete penetration is accepted and for the minimum quality control level 10% of 40/150 = 1.3%, or 10% of 100/150 = 6.6%. Therefore, these values are safe compared to the results of Ben Kato et al, especially because the strength of the base metals used is also partly outside the limits of Ref. 1.

For porosity and slag inclusions it is well known (see Refs. 8 and 9) the weakening is nearly equal to the reduction of the cross section: the line DEW, in Fig. 4. Therefore, even more than 20% could be fit for purpose. In Ref. 3, porosity up to 4% is allowed and slag inclusions up to about 10%—"Slag inclusions are allowed up to an overall length of not more than 6 in. (150 mm) per 6 in. (150 mm) of weld"—(minimum quality control level). For undercut, the maximum permissible value in the minimum quality control level is 10% for a plate thickness less than 0.4 in. (10 mm) and 0.04 in. (1.0 mm) for a plate thickness more than 0.4 in. (10 mm). Cracks and lack of fusion are not allowed.

From the above, and from many other tests[6] it may be concluded in case of plastic failure, the Code of Practice[3] gives safe values. It may be expected that even in less *ductile* weldments due to more or less incorrect choices for the base metal, the filler metal and/or the welding procedure, the Code of Practice[3] will not lead to dangerous situations.

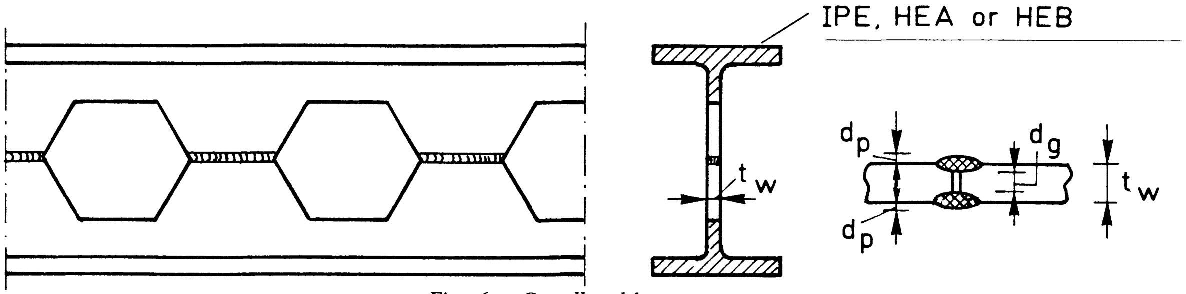


Fig. 6. Castellated beam

5. AN EXAMPLE OF "FITNESS FOR PURPOSE"

The weld in castellated beams can easily be tested by radiography.

This may be the reason why, in case of doubt, the owner sometimes takes the easy decision to make radiographs of the welds with the "worst" surface. The NDT firm then often finds porosity and incomplete penetration. This may lead to the IIW classification 4 (brown) or 5 (red). Sometimes it is stated in the contract that "the welds should be of good quality." Sometimes, "the welds should meet Class 2 of the IIW classification." It is clear that big problems will arise, especially when the welding shop is small; because then the huge repair costs can be a real burden.

When applying the Code of Practice,[3] the weld defects (incomplete penetration) may be found to be too long to meet the lowest level. In such a case:

- The webs of these beams usually have thicknesses between ¼ and ½ in. (6 and 12 mm). A continuous incomplete penetration of 0.08 to 0.16 in. (2 to 4 mm) will often be accompanied by a weld reinforcement of about the same magnitude. From this, it can be expected that the reduction in the strength of the weld due to defects will be compensated to a large extent by the reinforcements.
- But how is the owner to be convinced his welds with IIW Classification 4 and 5 are fit for their purpose and that expensive repairs are not necessary and only cause delay?

The best procedure in such cases is not through discussion, but in carrying out tests on pieces taken from the castellated beams. These test pieces are chosen so they contain the "worst" welds. For a particular case, Fig. 7 illustrates such a test piece (Fe 360 steel, σ_u = 52 ksi). In this test piece the load is perpendicular to the welds. In reality, the main load is in shear. But a test piece for shear is however not easy to make. If the test piece of Fig. 7 behaves well, then the welds will also behave well for a shear load. The IIW classification of the NDT for this example is given in the next table.

Number	IIW	Defects
18	5	Aa, Ab, C, D
20	5	Aa, Ab
22	4	Aa, D

With
Aa = Porosity
Ab = Worm holes
C = Lack of fusion
D = Incomplete penetration

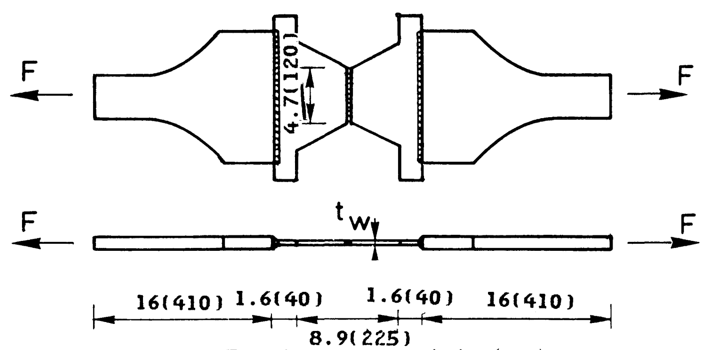


Fig. 7. Test piece, measures in in. (mm)

Results of the tests are shown in this table:

Number	Length of Weld in. (mm)	Thickness of Web in. (mm)	F_u kip (kN)	σ_u ksi (N/mm^2)	Failure pattern; see also photographs
18	4.7 (120)	0.24 (6.09)	70.8 (315)	62.5 (431)	
20	4.7 (120)	0.25 (6.24)	68.8 (306)	59.3 (409)	
22	4.7 (120)	0.24 (6.19)	65.0 (289)	56.4 (389)	

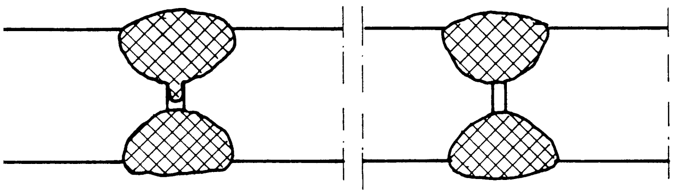


Fig. 8. Lack of fusion

As seen from this table, the failure was outside the welds and the ultimate stresses higher than the nominal values of σ_u = 52 ksi (360 N/mm^2) for Fe 360 steel for all test pieces.

The lack of fusion of weld No. 18 did not reduce the strength of the test piece. A closer inspection to the welds showed this lack of fusion was caused by some filler metal dropped between the root faces. This may occur in the CO_2 welding process used.

From Fig. 8 and the above table it becomes clear that, in this case, lack of penetration will not be harmful to the strength of the weld. The strength of weld (b) without lack of penetration is the same as that of weld (a).

6. CONCLUDING REMARKS AND RECOMMENDATIONS

1. Application of the Design Rules[1] and the Code of Practice[3] of Commission XV will lead to safe structures. The conditions in Refs. 1 and 3 regarding the selection of the quality of the base metal and the choice of the filler metal and parameters in the welding procedure are not very specific. Guidelines in this field are needed. For instance, the ECCS has already published guidelines for the selection of the quality of the base metal.[10] It is desirable for the IIW to publish guidelines for the combination, which includes the selection of the base metal and the choice of the filler metal and welding parameters.
2. The limitations of Refs. 1 and 3 are for steels having a nominal strength not higher than the strength of Fe 510 steel (σ_u = 74 ksi). After the publication of Ref. 1, higher strength steels are used more frequently. Is it possible to extend the scope of Refs. 1 and 3 to cover these higher strength steels? If IIW does not give guidance in this field, other bodies will make their own choices. See for instance Eurocode No. 3.[11]
3. As mentioned by many others before, the IIW classification is often misused and leads to time consuming discussions and unnecessary repairs. Furthermore, the information given in reports of NDT firms is usually not sufficient for the application of codes of practice on weld discontinuities. Often, this information does not go further than mentioning the discontinuities which were found (Aa, Ab, C, D etc.). It would be better to leave out the IIW classification and give the quantitative information which is needed for the application of the codes of practice on weld discontinuities.

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