A rational approach to standards for welded constructions

Concluding Part.

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IMPACT

There is sufficient evidence to indicate quite reliably that the effect of impact at the rates of loading normally expected in engineering even if they may be fairly fast - say the hammerblow of a locomotive on a bridge or the water hammer in a penstock - is no worse than ordinary, slow static loading. At explosive rates of loading the situation changes of course, but this is perhaps a rather specialised field which would not normally concern anybody but the military engineer. Even under impact loading at ordinary engineering speeds even fairly gross defects in welds in mild steel will not reduce resistance to fracture so that the criteria for accepting defects under conditions of impact need not differ from those applied to conditions of purely static loading provided, of course, that the material is not notch brittle at the temperature in question.

BRITTLE FRACTURE

It is in relation to the risk from brittle fracture that defects really become important. No case of brittle fracture in a welded structure has ever occurred that did not initiate from a weld defect.

Having said this it is important to emphasise that we need concern ourselves only with low stress brittle fracture, that is the risk of failure by brittle fracture at stresses lower than the design stress, since yield point fracture will never occur in practice. Stresses producing a state of general yield over a very large area of the structure will not normally occur unless a very serious mistake in design calculations has been made.

The distinction between yield point fracture and low stress fracture is important in relation to the role of defects. Yield point brittle fracture could initiate from a defect of sufficient size provided the material was at a temperature well below its Charpy V notch transition temperature whereas low stress brittle fracture requires in addition the presence of very high residual tensile stresses in the region of the defect. It follows that even fairly large crack-like defects perhaps equal in length to the plate thickness could be tolerated in a structure that had been stress relieved and that was not subjected to fatigue even if service temperature of the structure was below its transition temperature. To put it in another way , since for low stress brittle fracture to occur there must be :

(a) a crack-like defect (ordinary porosity not extended by cracking would not constitute a risk) ;

(b) high residual tensile stresses in the region of the crack, and

(c) a service temperature well below the Charpy V notch transition temperature.

The elimination of any one of these three factors would eliminate the risk of brittle fracture. That is to say, at a service (or test) temperature above the transition temperature even large crack-like defects (provided fatigue is absent) can be tolerated and stress relieving is unnecessary. In a stress relieved structure, defects can be tolerated even if the material is brittle and if serious crack-like defects are eliminated, stress relieving is unnecessary even if the material is brittle. One may go further still and say that stress relieving can be omited even in the presence of fairly large crack-like defects (not exceeding plate thickness in length), even in material that is brittle at the lowest temperature reached by the structure, provided that (a) the structure has been subjected to an overload test at a temperature well above the Charpy V notch transition temperature before being put into service, and (b) that, at temperatures below the transition temperatures, applied stresses are lower than those experienced in the overload test.

Experience has shown that only elongated crack-like defects present any real danger in relation to the initiation of brittle fracture. This is because only such defects produce strain concentrations of sufficient magnitude at their pointed ends under normal service loading to make brittle fracture initiation possible.

FATIGUE

Fatigue failure is probably the most common type of failure in welded construction, but only a relatively small number of all such failures experienced are the direct result of the type of weld defect - porosity, blow holes, slag inclusion, lack of fusion, etc. - that occurs accidentally and gives rise to the usual arguments between inspectors and contractors. Deliberate defects such as that which occurs when either no weld preparation for a butt weld is shown on a drawing, or a preparation which will permit welding only about halfway through the plate thickness (Fig. 7) are known to have caused fatigue failures in service.

The reason, of course, is obvious. In such a weld, which hardly deserves the name, the abutting surfaces of the unwelded part of the plate constitute an exceedingly sharp and deep fissure which under cyclic stressing of even quite low magnitude will propagate almost from the start and penetrate very rapidly through the weld. The incomplete penetration butt weld should be prohibited for all but non-load carrying structures. It is as dangerous in respect of brittle fracture as it is in respect of fatigue. Newman[2] showed that if this type of defect is present in circumferential butt welds of B.S. 806; Class B mild steel pipe of 6-5/8 in o.d. and 3/8 in wall thickness it would reduce the fatigue strength in reversed bending for 2 x 10 cycles to ± 1.1/4 tons/sq.in which is only between 15 p.c of the fatigue strength for unwelded pipe (±7.1/2 tons/sq.in. to ± 11 tons/sq.in. ; mean 8.1/4 tons/sq.in.) and 35 p.c. of the fatigue strength of sound butt welds (± 3.3/4 tons/sq.in.).

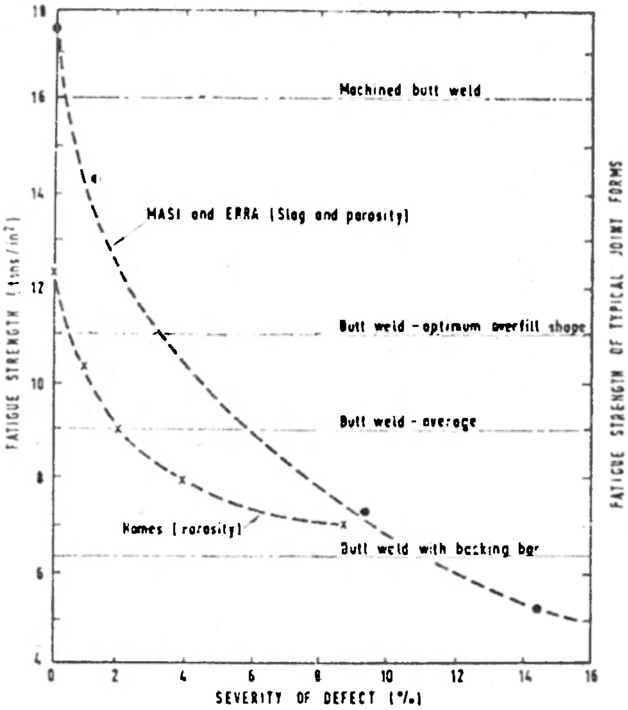

Fig. 8 — Effect of porosity on fatigue strength of butt welds. Porosity of 2 p.c. can be tolerated without detrimental effect on fatigue strength of unmachined butt welds, whereas porosity of only 1 p.c. would lower fatigue strength of machined butt welds (Mild steel).

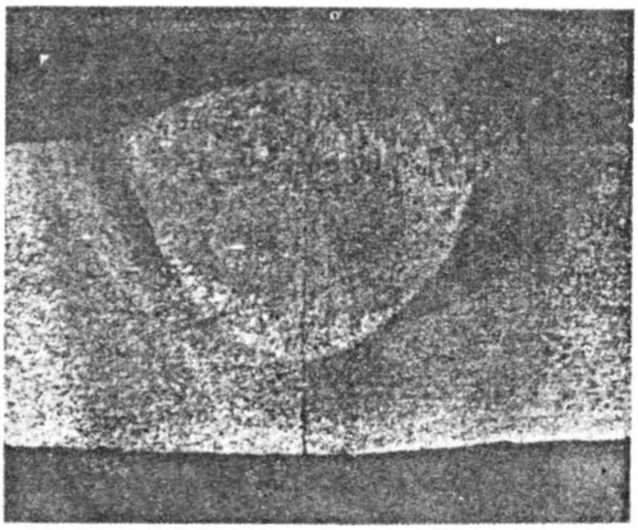

Fig. 7 — Incomplete penetration butt weld. Many fatigue failures and brittle fractures have been initiated by this deliberate defect designed into the structure. Such welds should never be permitted.

Most fatigue failures experienced in practice, with the exception of those due to this particular defect, stem from bad design of detail and this is one of the consequences of our using completely outdated methods of design. The fatigue resistance of a structure is determined by the magnitude of the stress at points of stress concentrations. Generally, in ordinary conventional design methods, we calculate only average stresses but ignore the stress concentrations due to such things as abrupt changes in section, sharp corners, attachments and other discontinuities. In a riveted girder for instance we calculate the maximum bending stress-making allowance for the loss of cross sectional area due to rivet holes - but we ignore the fact that the actual maximum stress in the cross section where the maximum bending moment occurs will be nearly trebled at the edge of each rivet hole. From experience we know, however, that using conventional methods of design

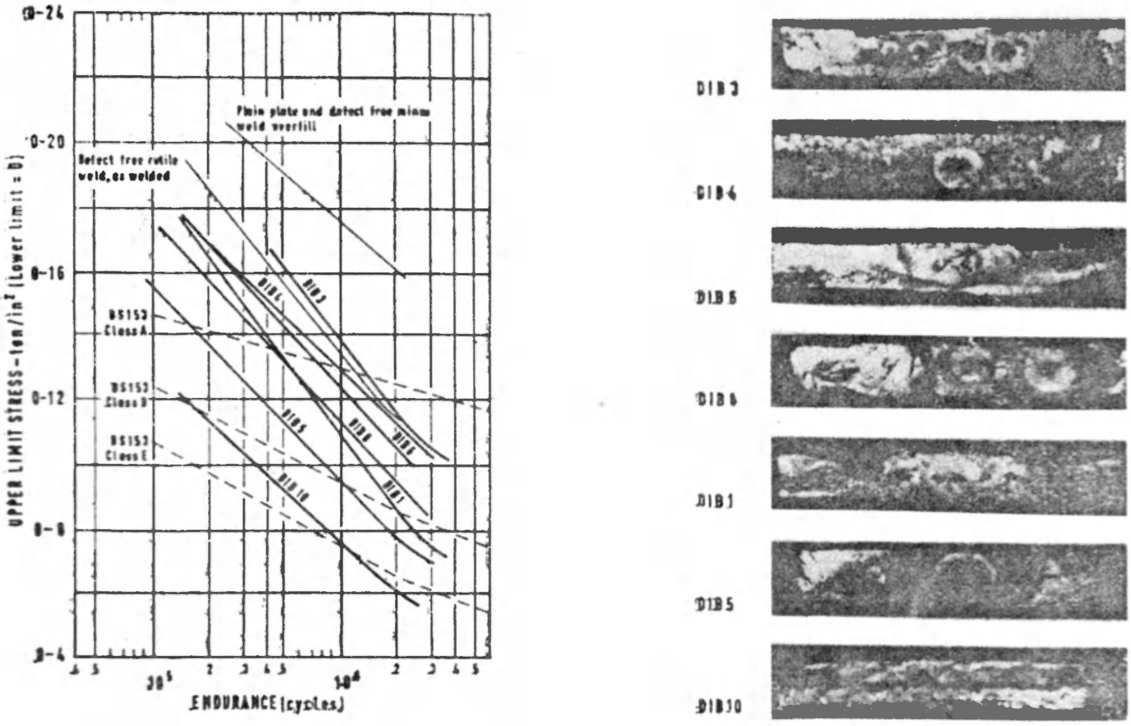

Fig. 9 — Fatigue S-N diagrams for butt welds tested in tension between zero and maximu The photographs on the right show the fatigue failures for different types of defect of ir severity (downwards) corresponding to S-N diagrams.

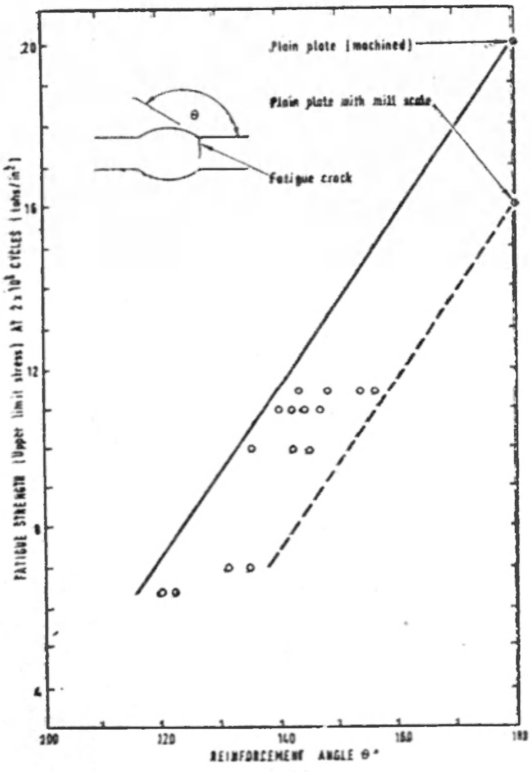

Fig. 10 — Fatigue strength of butt welds diminishes with decreasing angle included between tangent to overfill and plate surface.

and permissible stresses it is quite safe to ignore this local stress increase and that riveted girders with certain notable exceptions (such as the stiffening truss of the Manhattan suspension bridge in New York which failed from fatigue after 40 years) are reasonably immune from fatigue failure for a very large number of years.

Welded design, however, may introduce much more severe stress concentrations - and not by virtue of defects in the welds - than riveting and it is a result of this fact that fatigue failures in welded structures are so common. For instance, the fatigue strength for two million cycles of a welded plate girder with a continuous web to flange weld will depend on whether the flanges are welded to the web by continuous automatic welding or by manual welding with its inevitable stop-start points when the electrode is changed. As a result of the stop-start points the fatigue strength of the manually welded plate girder will be about 9 tons/sq.in. in terms of maximum bending stress in the flange and that of the automatically welded girder without stop-start points in the web to flange welds will be 11 tons/sq.in. to 12 tons/sq.in. A butt weld in the flange may reduce the fatigue strength to 7 tons/sq.in. and a transverse fillet weld attaching a stiffener to the flange will reduce the fatigue strength of the girder flange still further to about 5 tons/sq.in. It is obviously quite immaterial to argue about even a serious defect in the butt weld of the flange if right next to this butt weld there is a transverse fillet weld across the flange. Continuous lack of root fusion (DB10 in Fig. 9), the most serious defect by far, will reduce the fatigue strength of the butt weld to only 6 tons/sq.in. which is not as large a reduction as that produced by the transverse fillet weld.

Defects, in as far as they produce stress concentrations, lower the fatigue strength of welded joints., However, it does not follow from this fact that, in structures subjected to fatigue, defects in welds cannot be tolerated. The diagram in Fig. 8 illustrates the results of fatigue tests carried out on butt welds with differing degrees of porosity. Although the fatigue strength falls with increasing porosity the diagram shows that porosity less than 2 p.c. could be tolerated in an unmachined butt weld with the overfill left on, whereas, of course, porosity of only 1 p.c. would reduce the fatigue strength of a machined butt weld by 6 tons/sq.in. from 16 tons/sq.in. to 10 tons/sq.in.

What decides whether a defect can be tolerated is the actual stress existing at the point where the defect occurs. If the fatigue strength of the weld for the appropriate number of cycles that the structure is expected to survive, even with this defect, does not fall below the stress known to exist at the particular point there is no reason to remove the defect.

It must be remembered in this context that in design for fatigue one cannot use a safety factor by choosing a permissible stress which is only a fraction of the fatigue strength. If it is known that the fatigue strength of a butt weld free from defects and with the overfill machined off is 16 tons/sq.in. for two million cycles, there is no reason why this figure should not be used in design. Safety lies in the fact that failure will not take in less than 2 million cycles. If on the other hand a stress lower than 16 tons/sq.in. is used in design for some other reason than fatigue, a weld completely free from defects need not be insisted on. If one chooses

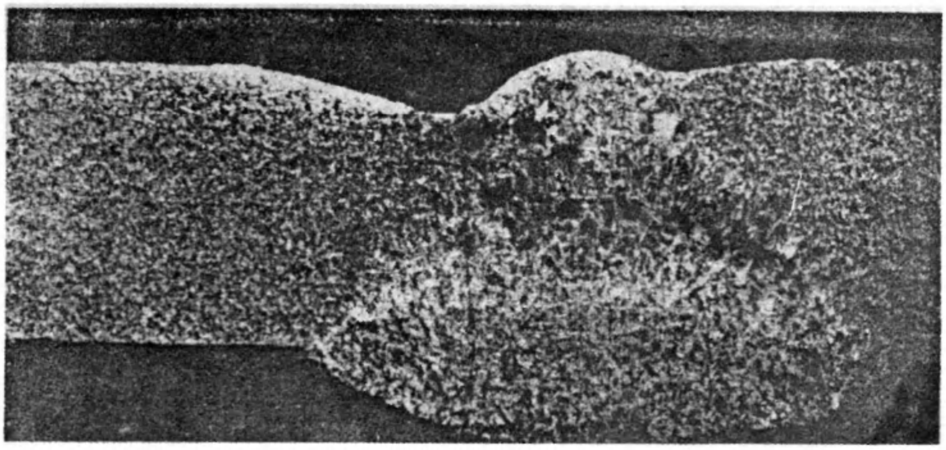

Fig. 11 — Fatigue crack in circumferential pipe weld starting from root overfill.

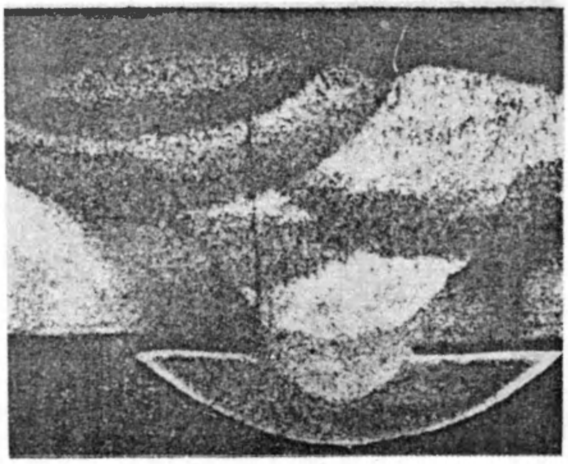

Fig. 12 — Fatigue crack in circumferential butt weld of pipe starting from notch between backing ring and pipe wall.

such high design stresses, one must, of course be certain that the number of cycles of maximum load has been correctly estimated for the life of the structure. Lower design stresses may have to be used, perhaps because the structure may very occasionally have to withstand an overload equal to twice the normal design load. This would not be expected to produce fatigue. If it were only experienced a few times during the life of the structure ; nevertheless in this case the design stress for normal loading might have to be reduced to 8 tons/sq.in. However, if it is, fairly severe defects may be tolerated in the butt weld as is evident in Fig. 9 (Newman and Gurney). On the right are shown the fracture surfaces of butt welds containing various types of defects typified by numbers DIB3 to DIB10 and on the left are given the S-N diagrams - that is, stress versus number of cycles to failure - obtained experimentally for these joints. Included in the diagram are two further lines giving the fatigue strength for defect-free welds both with the overfill left on and machined off. The three dotted lines give the permissible stress levels in the British Standard for Steel Girder Bridges (B.S. 153) for three classes of butt welds (see the accompanying Table).

Definition for Classes A, D and E butt welds (B.S. 153)

	A	D	E
(i)	Plain steel in the as-rolled condition with no gas cut edges.	Members fabricated with full penetration transverse butt welds having the weld reinforcement dressed flush and with no undercutting.	Members fabricated with transverse butt welds, other than previously mentioned, or with transverse butt welds made on a backing strip.
(ii)	Members fabricated with continuous full penetration longitudinal or transverse butt welds with the reinforcement dresse flush with the plate surface and the weld proved free from defects by non-destructive examination, provided also that the members do not have exposed gas cut edges. Welds shall be dressed flush by machining or grinding, or both which shall be finished in the direction parallel to the direction of the applied stress.	Members with continuous longitudinal fillet welds with start-stop position within the length of the weld.	Members fabricated with full penetration cruciform butt welds.
(iii)		Members of mild steel to B.S. 15 (mild steel for general structural purposes) or B.S. 2762 (notch ductile steel for general structural purposes) fabricated or connected with rivets or bolts.	

It will be seen that for Class A design stresses, perfect butt welds with reinforcement machined off are essential but that all manner of defects - except continuous lack of fusion - are acceptable for Class E for 2 million cycles. For less than 2 million cycles even that defect is acceptable at a much higher design stress.

It is particularly irritating in relation to the whole argument about defects that it focuses attention on defects revealed by non-destructive testing - porosity, slag inclusions, etc. - but completely ignores those that one can see just by looking at the joint, but which, for no earthly reason, are assumed not to matter.

In another investigation carried out by Newman and Gurney on the effect of the angle formed between the plate surface and the overfill they have shown (Fig. 10) that the fatigue strength of perfect, defect-free butt welds may vary between 6 tons/sq.in. for an angle of 120° to 11 tons/sq.in. for an angle of between 140° to 160°. Comparing these figures with the S-N curves in Fig. 9 it is obvious that if such an acute angle as 120° is tolerated for any butt weld one can tolerate also at the same time the continuous lack of root fusion defect without reducing the fatigue strength to below 6 tons/sq.in. and even for an angle of 160° one can still tolerate some fairly hefty slag inclusions.

Newman in the investigation on the fatigue strength of butt welds in pipes already mentioned found that fatigue failure invariably started from the root of the weld (Fig. 11 and Fig. 12) irrespective of whether or not a backing ring was used and whether the root of the weld was machined or not. Any additional defects, even such gross defects as those shown in Fig. 13, had no effect in lowering the fatigue strength any further. In one particular case where fatigue failure started from the root of the weld the lack of sidewall fusion defect shown in Fig. 14 was found after the broken specimen was examined and even this very serious defect could not compete with the overriding effect of the stress concentration in the perfectly sound and normal root of the butt weld.

In assessing the fatigue reducing effect of defects in welds one must consider first and foremost the overall fatigue strength of the structure or component. Fatigue failure will start from the most sensitive point and this may not be - and very rarely is - a weld defect. The rolled surface of the plate itself produces a reduction in fatigue strength from 20 tons/sq.in. to 16 tons/sq.in (Fig.10), the overfill of the butt weld produces a further reduction and so will any trace of undercut. What stress concentration effects may be produced by defects may be simply swamped by the effect of other stress concentrations.

If however, there are no other stress concentrations than those due to the weld defect even a single small sub-surface pore may produce fatigue failures and reduce fatigue strength. In a very extensive investigation of the problem where machinery shafts reclaimed by welding frequently fail from fatigue very soon after they have been put into service. Dawes found that fatigue failure in 0.32 p.c. C steel shafts reclaimed by welding and machined would take place (at ten million cycles) at between ± 9.5 tons/sq.in. to ± 11.5 tons/sq.in. rotating bending fatigue stress, which he estimated to be only about 65 p.c. of the fatigue strength of the original machined but unwelded shaft, and that fatigue failure would originate even from a single small pore as that shown in Fig. 15.

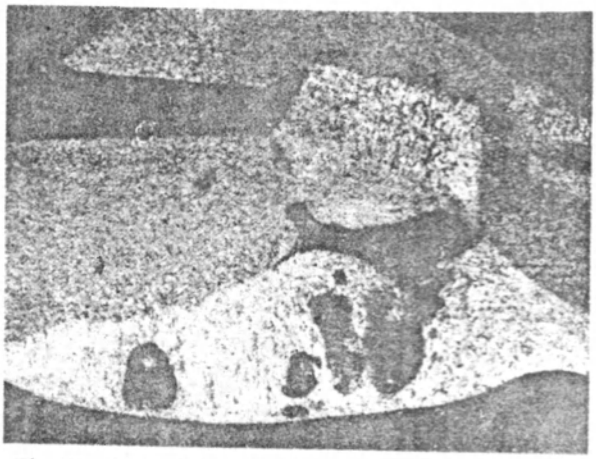

Fig. 13—Gross defects in circumferential butt weld in pipe had no effect on bending fatigue strength. (Pressure strength may, of course, be reduced.)

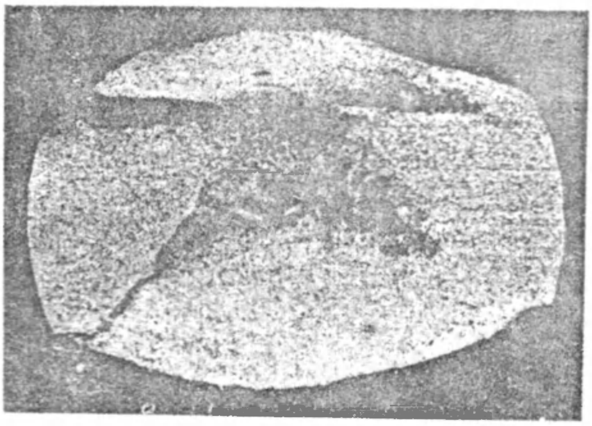

Fig. 14—Lack of sidewall fusion in circumferential butt weld in pipe had no effect on bending fatigue strength of pipe. Fatigue failure started from perfectly normal and acceptable weld root.

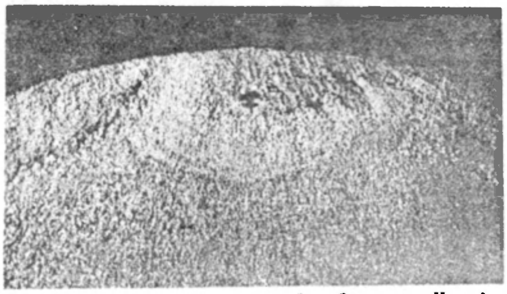

Fig. 15—Fatigue failure starting from small subsurface pore in experimental shaft reclaimed by welding the shaft prior to "reclamation" was, of course, virgin material and had not suffered any previous fatigue.

This is not really surprising when it is remembered that a small pore will locally produce a stress concentration of nearly three and that if this is the only stress concentration present in the whole of the shaft fatigue failure will start from this pore and the fatigue strength of the shaft will be reduced, though as will be observed not to a third of the strength i.e. not proportionately to the stress concentration factor.

A few examples presented in this survey illustrate the complexity of the general problem of tolerance levels for defects in welded structures. The subject is more complex even that has been indicated because the position of the defect in relation to the surface of the joint is important in joints made in thicker material. The location of defect in a field of residual stresses, whether tensile or compressive, may influence the effect of the defect both on fatigue strength and brittle fracture strength. However, to make decisions on accepting or rejecting welds by ignoring all the experimental information now available is most unscientific and puts the engineer in the same class as the medicine man of a primitive tribe. Admittedly a great deal more experimental information is needed but enough is available already to show that the importance attached to certain types of defect is frequently grossly exaggerated and may in fact divert attention from other undesirable features in design and execution.

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