FATIGUE ASPECTS IN STRUCTURAL DESIGN

Edited by
J. Wardenier
J.H. Reusink
FATIGUE ASPECTS IN STRUCTURAL DESIGN
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edited by
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The valedictory address (in Dutch) by *prof. J. de Back* is not included in the proceedings.
This symposium on “Fatigue Aspects in Structural Design” is organised to honour Prof. ir. J. de Back on the occasion of his retirement on October 1, 1989.

An important part of his research work is devoted to fatigue of steel structures. Various programmes on fundamental and practical aspects have been carried out in the Stevin Laboratory of the Delft University of Technology. The results of these programmes have been implemented in national and international design codes and found their way in the practical design of bridges, cranes, offshore structures and rotor blades for energy windmills.

Many research programmes were carried out in a broad international cooperation, especially those for the ECSC (European Coal and Steel Community). ECSC Working Group III, chaired by prof. de Back, had a leading role in the field of fatigue research for offshore structures. The close cooperation with specialists in the ECSC, the ECCS (European Convention for Constructional Steelwork) and the IIW (International Institute of Welding) had a stimulating effect on the developments in fatigue design.

The authors of the papers of this symposium are leading experts in the “fatigue” world. The first part of this symposium deals with the more fundamental aspects whereas the second part covers the evaluation to design rules and the current developments in the various parts of the world.

A good interaction between the fundamental side and practical application, together with a broad international cooperation, will ensure optimal results. Prof. de Back is one of the specialists who has combined these aspects.

Prof. dr. ir. J. Wardenier
Chairman of the symposium

Delft, September 1989
SIGNIFICANCE OF WELD DEFECTS WITH REGARD TO FATIGUE BEHAVIOUR

J.D. Harrison


SUMMARY

The paper describes the advantages of using a fitness-for-purpose approach to assess the significance of weld imperfections with regard to fatigue performance. An International Institute of Welding recommendation is outlined which gives a basis for such assessments. Normal weld design details, such as fillet welds, have relatively low fatigue strengths and apparently severe weld imperfections may not reduce fatigue strength by as much as these details. The paper closes with a series of case histories.

1. INTRODUCTION

Clearly any product must be fit for its purpose. However, in the field of welding, the term "fitness-for-purpose" has come to be applied to methods for reaching repair/no repair decisions concerning imperfections found by non-destructive testing (NDT) during manufacture or in service.

Most specifications for high integrity structures require that some or all of the welds be inspected by NDT. Imperfections found then have to be assessed. The assessment criteria are usually either arbitrary or based on "good-workmanship". This would be reasonable, were it not for the fact that imperfections which fail to comply are required to be repaired. The vast majority of repairs are carried out for imperfections which could have no conceivable harmful influence on subsequent performance [1,2]. Indeed, the repairs are often more deleterious than the imperfections themselves. This is illustrated by a survey of the service performance of several large ships [3]. From this, the following facts emerged:

a. NDT prior to delivery was confined to a small percentage of the butt welds in the hulls.

b. Imperfections found during this partial inspection were repaired, if they lay outside the arbitrary criteria.

c. Fillet welds were not subjected to NDT.

d. Most service cracks initiated at fillet welds!

e. Occasionally service cracks occurred in butt welds; but, when they did, it was almost invariably at a repair!

A further problem with repair welds is that, because they are often made under more difficult conditions of access, etc., than the original, as many as 20% may require re-repair [1]. This repair and re-repair for harmless imperfections is farcical.

Quite apart from their possible harmful effects on service performance, repairs are expensive. Grant and Rogerson [1] showed that, for certain
offshore equipment modules, the man hours for repair were 9% of total construction time. However, the direct costs can be swamped by the consequential costs in terms of late delivery, lost production, etc. Actual weld repair costs in shipbuilding were estimated by Johansen et al [3] in 1981 at $10-30K per super tanker built in a Swedish yard. However, this excluded all consequential costs. Sandor [4], also writing in 1981, estimated the total cost of weld repair, including consequential costs, at $0.6-1.0M per ship built in US yards.

The author is strongly of the opinion that combining NDT with arbitrary acceptance standards is most undesirable. It is costly and often detrimental to safety. It is much better to give NDT the dual rôle of quality control (QC) and acceptance. In its QC rôle, NDT may be used to draw attention to the need to adjust the welding procedure. Repairs are only required if the imperfections found might impair fitness-for-purpose. Such an approach has the following desirable features:

i. It can reduce the number of repairs significantly; often by one or two orders of magnitude.

ii. It concentrates the attention of designers and welding engineers on those aspects of a structure which contribute most to integrity; be they stress level, material properties or imperfections.

Some codes encourage this approach. For example, the British pressure vessel and pipeline codes [5,6] both give arbitrary acceptance levels for weld imperfections. Provided there are no flaws exceeding these, the weld can be accepted. However, imperfections which do exceed these levels are not an immediate cause for rejection. They can, by agreement between the parties, be assessed in terms of fitness-for-purpose.

Such a procedure calls for a framework for assessment. This was proposed in the UK in the late 1960s [7,8] developed during the 1970s and issued as a Published Document, PD 6493 [9], by the British Standards Institution early in the 1980s. The International Institute of Welding (IIW) has undertaken a parallel initiative. Four Commissions, V - NDT, X - Fracture, XIII - Fatigue and XV - Design, are collaborating to produce an ISO document giving the basis for an engineering critical assessment (ECA) of weld imperfections.

Clearly the whole basis of an ECA is to assess the significance of an imperfection with reference to all the potential failure modes. PD 6493 identifies the following:

Fracture
Yielding of remaining section
Buckling
Creep

Fatigue
Leakage
Environmental cracking

This paper is concerned with the significance of weld imperfections with regard to fatigue. The introduction has set the background for a need for such information.

2. EARLY STUDIES

Most of the early investigations concerned the effect of weld imperfections of the types found by radiography (porosity and slag inclusions), that being the most common form of non-destructive testing at the time.
It was soon evident that welds were very tolerant to such imperfections. Apparently severely defective welds could have fatigue strengths matching those of the original design details. One such early study was by Newman [10], who examined the fatigue performance of pipe butt welds made onto backing rings. Figure 1 shows a section through a joint which he described as "grossly defective". Few would quarrel with that description! Yet fatigue failures were not associated with the "defects" but occurred from the edge of the backing ring and the fatigue results lay within the scatter band for the nominally defect-free control series (Fig. 2).

Research continued in the 1960s and 70s and from 1961, Commission XIII of the IIW, under the Chairmanship of Monsieur de Leiris, co-ordinated various studies. One such programme involved research into the significance of slag inclusions. Specimens, with a variety of lengths, numbers and orientation of inclusions, were welded in England and distributed for fatigue testing to collaborating laboratories in West Germany, France, Jugoslavia and the Netherlands. This co-operation made it possible to generate large volumes of data. The results were reported by Harrison [11] and later analysed statistically by Harrison and Doherty [12].

The approach used to study the effect of slag and porosity on fatigue behaviour was to generate series of S-N curves which related to various degrees of imperfection. The advent, in the mid-1960s, of the application of fracture mechanics to fatigue [Paris and Erdogan [13]] gave a better method of dealing with crack-like imperfections.

These researches, along with those relating to fracture, all came together in the late 1960s and early 70s and formed the basis for the early drafts of PD 6493. The fatigue section of this document was accepted by Commission XIII at the IIW Annual Assembly in 1978 and was transmitted to ISO for information. The background of this fatigue section was described by Harrison [14]. Finally, within the IIW a proposed guidance document was put to a joint meeting of the four collaborating Commissions at the Annual Assembly in Vienna [15]. The following sections of the present paper describe the fatigue section of that document (Part 4).

3. FATIGUE DESIGN

Before discussing the effect of weld imperfections on fatigue behaviour, it is worth mentioning the fatigue design of nominally sound welds. The IIW has produced recommendations [16] based on a study of the world literature giving S-N data for welded joints. These recommendations are based on a series of parallel and equally spaced log S - log N curves (Fig. 3) with a slope of $-\frac{1}{3}$. Thus each curve has the form $S^3N = \text{Constant}$. These curves are characterised by the stress range in N/mm$^2$ (for steel) corresponding to an endurance of $2 \times 10^6$ - the notch Class. The design recommendations incorporate notch Classes 45 to 125. The recommendations then tell the user into which notch Class all the various types of weld detail fall.

4. FITNESS-FOR-PURPOSE ASSESSMENT OF WELD IMPERFECTIONS UNDER FATIGUE LOADING - THE IIW RECOMMENDATION

4.1. Background

Fatigue cracks can originate from planar (cracks, lack of fusion, etc.), or volumetric (slag inclusion, porosity) flaws. In the IIW Recommendation, fracture mechanics principles are used to describe the behaviour of
planar flaws, whilst the assessment of volumetric flaws is based on experimental S-N data. Guidance is also given on the assessment of shape imperfections (misalignment, angular distortion, undercut). The Recommendation can be used to assess the acceptability of known flaws or to fix tolerable flaw sizes prior to construction.

It should be noted that, quite apart from weld flaws, the very presence of a weld will reduce the fatigue strength to levels which may be substantially below the fatigue strength of the unwelded material. Weld flaws will only be significant with regard to fatigue, if they reduce the fatigue strength below the inherent strength imposed by the presence of any other weld lying close to the flawed weld under consideration.

Most fatigue failures in structural-type steelwork (ships' hulls, cranes, earthmoving and mechanical handling equipment, bridges, etc.) result from faulty design. They initiate at the toes of fillet welds and it is very difficult to avoid the use of these in such structures. Failures are not associated with weld defects in the normally accepted sense of the term; but initiate from small slag intrusions which are trapped at the toes of all welds [17,18]. The predominance of fillet welds as initiation points for service fatigue failures was shown by the present author [19] in an analysis of failures reported to the International Institute of Welding [20,21]. Usually failure occurred because the stresses were simply too high and the designer had failed to take account of the relatively low fatigue strength of fillet welds. Adherence to any of the published fatigue design rules [16,22-24] would have prevented the failure.

Weld discontinuities other than those at the toes of fillet welds have to be quite severe to reduce the fatigue strength to that of a fillet weld.

4.2. Methods used to assess various flaw types

4.2.1. Fracture Mechanics Analysis of Planar Flaws

The fracture mechanics approach is based on the observed relationship between the range in the stress intensity factor, $\Delta K$, and the rate of growth of fatigue cracks, $da/dN$. This usually takes a sigmoidal form in a log $\Delta K$ versus log $da/dN$ plot (Fig. 4). Below a threshold stress intensity factor range, $\Delta K_0$, no growth occurs. For intermediate values of $\Delta K$, growth rate is idealised by a straight line in the log/log plot such that:

\[
\frac{da}{dN} = A(\Delta K)^m
\]  

When the maximum stress intensity in the cycle approaches the critical value for fast fracture, rapid acceleration of the crack occurs and this forms the upper part of the sigmoidal curve.

Fracture mechanics analysis enables $\Delta K$ to be expressed in terms of the instantaneous crack size, $a$, the stress range, $\Delta \sigma$, and a parameter, $Y$, which is itself a function of the crack size and the geometry

\[
\Delta K = Y \frac{\Delta \sigma \sqrt{a}}{Y}
\]
The overall life absorbed in growing a crack from an initial size, \( a_i \), to a final size, \( a_f \), can be determined by integrating eq. 2

\[
\int_{a_i}^{a_f} \frac{da}{Y_m(\sigma_a)^{m/2}} = A(\Delta \sigma)^m N
\]  

(3)

Knowing the stress range, \( \Delta \sigma \), and the final crack size, \( a_f \), eq. 3 can be used to assess the acceptability of a crack of size \( a_i \), depending on whether the predicted life, \( N \), is greater or less than the design life.

4.2.2. Assessment of Volumetric Flaws

This is performed in terms of S-N curves obtained by statistical analysis of relevant test data, as described in more detail later.

4.2.3. Misalignment and Angular Distortion

These cause secondary bending in welds unless they are restrained laterally. This bending can be taken into account by calculating an appropriate stress magnification factor, \( K_m^* \).

4.2.4. Undercut

Like volumetric flaws, weld toe undercut is assessed by reference to experimentally determined S-N curves.

4.3. Data Required

4.3.1. Stress

Total stress range only is used, no account being taken of the ratio of minimum applied stress to maximum applied stress. This is because the effect of this ratio is nullified by the presence of tensile residual stresses in the weld areas. This is so even for post weld heat treated (PWHT) components because residual stresses remaining after such treatment are still significant compared to normal fatigue stresses. Most structures are subjected to a random distribution of stress ranges. In such cases, one needs to know the stress range spectra.

4.3.2. Flaw Dimensions

Planar flaws are idealised in terms of the length and depth of rectangles which would contain them, these then being translated into the major and minor axes of an ellipse (buried) or semi-ellipse (surface), for the purposes of fracture mechanics analyses.

Slag inclusions are characterised by their length and porosity by the percentage area on a radiograph.

4.3.3. Crack Propagation Data

Information is required concerning the values of \( AK_0 \) and of the crack growth parameters, \( A \) and \( m \). These may be obtained by specific tests on the material of interest or by information gathered from the literature. Account must be taken of aspects such as environment, cyclic frequency, waveform and residual stress, all of which affect growth rates.
Steels

In the absence of specific data, the Recommendation proposes that, for ferritic steels, upper limits on growth rate are given by:

\[
m = 3 \\
A = 3 \times 10^{-13} \text{ for non-aggressive environments at temperatures up to } 100^\circ C. \\
A = 3 \times 10^{-12} \text{ for marine environments at temperatures up to } 20^\circ C.
\]

The Recommendation also proposes an approach for determining the threshold, \( \Delta K_0 \), if specific data are not available.

For many practical situations in steel this reduced to the simple statement that \( \Delta K_0 = 63Nmm^{-3/2} \).

Non-Ferrous Metals

It is observed that both the crack growth rates and the thresholds in non-ferrous alloys are similar to those in steel for a given value of \( \Delta K/E \), where \( E \) is Young's Modulus. Thus, whilst specific data for the material and environment are to be preferred, in their absence the following are a reasonable basis for assessment:

\[
m = 3 \\
A = 3 \times 10^{-13} \left( \frac{E_{\text{Steel}}}{E} \right)^3 \\
\Delta K_0 = \Delta K_{0\text{Steel}} \left( \frac{E}{E_{\text{Steel}}} \right)
\]

4.3.4. Limits to Crack Propagation

In the fatigue assessment of planar flaws, the final crack size, \( a_f \), is fixed by the intervention of some other failure mode (fracture, yielding, leakage, etc.). Other parts of the Recommendation describe how this should be determined.

4.3.5. Probability of Survival

The Recommendation generally uses data appropriate to the mean minus 2 standard deviations (97.7% probability of survival). If the user wishes to adopt a different value, he can take advantage of the fact that the steps between the grid of S-N curves (Fig. 3) are approximately equivalent to one standard deviation.

4.4. Assessment

4.4.1. Planar Flaws

Two approaches to the assessment of planar flaws are given in the Recommendation, the general procedure and the simplified procedure. Both employ an integration of the crack growth law.
General Procedure

The general procedure calls for a specific crack growth analysis. Such analyses can be carried out with relative ease on a micro-computer. The Recommendation gives methods of calculating the parameter, $Y$, in eq. 2 for a variety of situations. One problem is that the stress intensity factor range, $\Delta K$, increases as the crack grows and $Y$ is a complex function of overall geometry and of crack size and shape. An analytical solution for the integral in eq. 3 can seldom be derived. The Recommendation proposes that the analysis should be done cycle-by-cycle. In practice this would be extremely laborious. An alternative is to break the crack growth down into a sufficiently large number of short steps of crack increment. Values of $\Delta K$ are calculated at the ends of each step. The number of cycles for the crack to extend by this step is calculated from this value of $\Delta K$ assuming it to be constant throughout the step. Because $\Delta K$ increases with crack size, it is conservative to use the value at the end of each step. The total number of cycles for all the steps is summed and compared with the required endurance. The Recommendation gives methods for allowing for the change in crack shape which occurs as a result of the different rates of growth at the ends of the major and minor axes of the ellipse or semi-ellipse.

Under random loading, low stress ranges, which, for the instantaneous crack size, lead to values of $\Delta K$ below $\Delta K_0$, are assumed to cause no crack growth. Values of $\Delta K$ above $\Delta K_0$ are assumed to imply crack growth in accordance with eq. 1. This assumes an idealisation of the bottom half of the sigmoidal crack growth relationship into two straight lines. See Fig. 4. If there are large numbers of cycles near the threshold, this idealisation can be excessively conservative. The IIW Recommendation gives an alternative somewhat less conservative, analysis which takes account of the curvature in the connecting portion of the log da/dN versus log $\Delta K$ relationship.

Simplified Procedure

In the simplified procedure the necessary integrations, etc., have all been performed for the user.

It employs the same grid of S-N curves as are used in the IIW fatigue design recommendations (Fig. 3). For the present purpose, they are labelled as Quality Categories. The lowest Design Notch Class is 45. The Quality Categories extend down to 20, to allow imperfections to be assessed in structures which have a lower level of fatigue loading than the maximum permitted for design purposes.

The required Quality Category is determined by entering Fig. 3 with the design stress range and the required endurance. The category is the next above the point so determined. The Recommendation gives a method for fixing the required Quality Category, when the structure is subjected to variable amplitude loading. This method is tantamount to the use of Miner's law.

The axis on the right of Fig. 3 is for aluminium alloys. This is the same as the left hand axis for steels, with the stresses divided by 3. This is based on the observation, already referred to, that growth rates in different materials are similar for similar values of $\Delta K/E$ and fatigue endurance similar for similar values of $\Delta c/E$. The ratio of the Young's modulus of steel to that of aluminium is 3.
The procedure of entering Fig. 3 at the design stress and required endurance assumes that these are known to the designer, as indeed they should be. However, an alternative rationale is to say that an imperfection is acceptable, provided it does not lower the fatigue strength below that of the original design detail. This is the advantage of using the same grid of S-N curves for the design notch Classes and for the Quality Categories.

Having fixed the required Quality Category, it is necessary to determine the actual Quality Category implied by the flaw under consideration. This is done using a series of curves which are given in the Recommendation. The curves cover a variety of weld details, flaw locations, axial and tension loading, attachment sizes, etc.

A sample is shown in Fig. 5. The first step is to convert the actual flaw dimensions in terms of the depth, \( a_1 \), for a semi-elliptical surface crack, or \( 2a_1 \) for a buried crack, and length \( 2c_1 \), into an effective initial flaw parameter, \( \tilde{a}_1 \), corresponding to a straight fronted crack (\( a/2c = 0 \)) using diagrams such as Fig. 5a.

A maximum tolerable flaw parameter, \( \tilde{a}_m \), to which fatigue crack growth can be permitted is also estimated. Note that, provided \( \tilde{a}_m \gg \tilde{a}_1 \), as it usually is, the precise value of \( \tilde{a}_m \) will make little difference to the outcome. In fact, in many cases, it will be quite satisfactory to assume that \( \tilde{a}_m = \) plate thickness.

The corresponding quality category is found from diagrams such as Fig. 5b, as follows:

i. Entering the figure at \( \tilde{a}_m \), on the ordinate axis and the thickness, \( B \), on the abscissa, read off a value of \( S (S_m) \), interpolating as necessary between the quality category curves.

ii. Similarly, determine \( S_1 \) by entering the diagram at \( \tilde{a}_1 \) and \( B \).

\[
S = (S_1^3 - S_m^3)^{1/3}
\]

What this procedure effectively does is to subtract the integral of eq. 3 for growth from \( \tilde{a}_m \), to \( B \) from that for growth from \( \tilde{a}_1 \) to \( B \), thus deducting the life lost as a result of the intervention of an alternative failure mode.

The actual Quality Category for the flaw in question is the next below \( S \) in Table 1. If this is the same as or higher than the required category the flaw is acceptable.

The curves in the Recommendation are based on crack growth integration assuming:

\[
A = 3 \times 10^{-13} \quad \text{and} \quad m = 3
\]

for crack growth in steel measured in mm/cycle and stress intensity factors in Nmm\(^{-3/2}\). This results in \(97\%\) probability of survival.
Note that, as stated above, this value of A is appropriate to growth in air at ambient temperatures. The Recommendation gives a method for determining the change in Quality Category which would occur as a result of a change in A, for example to that appropriate for seawater.

The curves of Fig. 5b highlight a thickness effect whereby, after an initial rise, the allowable initial flaw size falls rapidly as thickness increases. The physical explanation for this is that the depth below the surface, over which the stress concentration effect of a fillet weld is active, is proportional to thickness. Thus, in a thick plate, the fatigue crack will be influenced over much more of its growth by this stress concentration. Fatigue endurance data support a thickness effect and indicate that the trend shown in Fig. 5b is reasonable.

4.4.2. Volumetric flaws

Surface breaking volumetric flaws should be treated as cracks.

Buried flaws are assessed by reference to Table 11. The limits given in this table are based on large volumes of published data for slag inclusions in steel welds [Harrison and Doherty [12]] and for porosity in steel and aluminium alloy welds [Harrison [27]].

Table 11 gives different limits for slag inclusions in as-welded and PWHT welds in steel. This is because PWHT removes hydrogen from the inclusions. The limits for porosity which could be permitted, purely from a fatigue point of view, are extremely large. The levels set in Table 11 are based on the density of porosity which might interfere with radiography and mask other flaws.

4.4.3. Shape imperfections

Misalignment

Misalignment and angular distortion cause an increase in stress due to secondary bending. However, this does not apply to welds loaded parallel to their length or to welds loaded in bending. Also, in many situations, there is restraint in a plane perpendicular to that of the welded plates. For example, secondary bending in a misaligned butt weld in the flange of an I-beam will be resisted by the web of the beam. The Recommendation gives a number of solutions for the stress magnification factor, \( K_m \), for specific situations, including misaligned butt welds, misaligned cruciform joints, "roof topping" in the longitudinal seams of pressure vessels, etc.

The acceptable level of misalignment for an otherwise sound weld can be determined by multiplying the nominal imposed stress range by \( K_m \), and entering Fig. 3 with this elevated stress range and the design life, N. If the point so determined is below the S-N curve for the appropriate Notch Class for the design detail in question according to the IIW fatigue design recommendation [16], the misalignment is acceptable. If the weld contains other imperfections, these can be assessed as set out in the foregoing sections but with an additional bending stress due to the misalignment.
Undercut

Acceptance levels for undercut are expressed as a proportion of the thickness, B. They were derived from a review of published data by Petershagen [28]. Because of the restricted database, they are limited to 10 mm < B < 25 mm and to a maximum undercut depth of 1 mm. Outside these ranges, the undercut should be assessed as a planar defect. For many practical situations, undercut depth is limited to 10% of thickness.

4.5. Additional Information

The Recommendation covers both the situation where a known defect needs to be assessed and where tolerable sizes of flaw are to be fixed.

There are two Appendices to the Recommendation. The first is a valuable compendium of stress intensity factor solutions for a variety of flaws in butt and fillet welds. These are presented in the form of equations to facilitate entry into a computer. In this context, it is worth noting that the Welding Institute is in an advanced stage of developing a computer based text animator - a quasi-expert system - which will take the user through BS PD 6493, performing all the essential calculations for him.

The second appendix describes two alternative numerical integration methods. One uses a number of finite crack growth increments. If a large number of steps (e.g. 1000) are used, these steps can be of equal size. Alternatively, if a smaller number of steps is employed (e.g. 15), they should be distributed logarithmically such that the size of the step grows as the crack grows in a similar manner to the increase in crack speed. The second method breaks the stress history into blocks and calculates the crack growth induced by each succeeding block. This method is accurate as long as each block length is less than 0.1% of the total cyclic life.

5. CASE HISTORIES IN THE APPLICATION OF FITNESS-FOR-PURPOSE CONCEPTS TO THE ASSESSMENT OF WELD IMPERFECTIONS

5.1. Introduction

The general concepts which underlie both BS PD 6493 and the IIW Recommendation have now been applied for a number of years. Many successful case histories could be cited. The following are some examples from the authors' experience. Some of them are described in greater detail elsewhere [29].

5.2. Offshore Structures in the North Sea

Fracture mechanics has been applied in one form or another to the following 20 major structures:

- Beatrice
- Beryl A & B
- Brae B
- Buchan
- Claymore
- Forties A-D
- Hutton
- Murchison
- Minian South
- North Cormorant
- Piper
- Tern
- Thistle A
- Troll
- Ula
- Vessel Frik

Specific aspects of the approach are illustrated by the following cases. Three relate to aspects of node construction. Figure 6 is a general illustration. The shaded portion (the node) is made as a sub-assembly and is
often post weld heat treated (PWHT) prior to erection into the platform. Thus, Welds A, B and C are all made in the shop. Welds D and E are site erection welds. Either the brace-to-brace stub welds (8) are made from the outside only or access to the root is provided by means of a window. In the latter case, however, the window has to be closed by a single-sided weld (F).

Forties Field Platforms

The nodes of the four Forties Field jackets, the first major structures to go into the northern North Sea, were PWHT prior to erection. The decision to heat treat was based on fracture mechanics, since it was found that as-welded weld metals were insufficiently tough to tolerate the imperfections which might occur. The decision was expensive, costing British Petroleum (BP) some £2M. The payoff came when, late in construction, substantial lack of side wall fusion defects (up to 25mm deep x 1m long) were found in the brace stub-to-chord welds (A) of one of the jackets. Repair was difficult and, had this been insisted upon, the summer weather window for launch would have been missed. A fitness-for-purpose analysis showed that fatigue crack growth would be minimal and the structure was accepted without repair. The delay which repair would have necessitated would have cost BP £6M per day in lost revenue.

North Cormorant

This had a similar history, though in this case the imperfections found at a late stage were transverse weld metal hydrogen cracks (chevron cracks) in the longitudinal seams (B) of the 100mm thick chords. Some 20% of all nodes were affected and in these seams there were cracks about every 25mm. Repair would have had horrendous consequences for delivery. Once again the imperfections were assessed and found to be innocuous. The platform was accepted. Shell estimate that the delay caused by repair, had it been required, would have cost them £60M.

Ninian South

The above two cases refer to imperfections found during fabrication. The Ninian platform suffered fatigue failure of a brace from the root of a single-sided window closure weld (F). The repair was extremely costly and, unfortunately, there were window welds in virtually every brace, each of which might be an initiation site for a further failure. Fracture mechanics analysis was used to narrow the problem down to the welds which were most severely loaded and then to determine the sizes of imperfections that could be tolerated in the root. This enabled underwater inspection to be carried out in the most cost effective manner.

Hutton

This was the first large floating structure, a tension leg platform (TLP). Weight saving was crucial. In this case detailed fracture mechanics analyses were performed at the design stage as a back-up to conventional design.

Buchan Alpha

This Pentagon semi-submersible was a sister to the ill fated "Alexander L. Kielland". BP wanted to moor it permanently on their Buchan Field. The Kielland was lost as a result of fatigue failure of an underwater hori-
zontal brace which was flooded. Because of this, it was decided that these braces on Buchan Alpha should be dry. However, it was necessary to use fracture mechanics to demonstrate that a through-thickness crack would remain stable, so that incipient failure could be detected by leakage into the brace, and that crack growth rates would be slow enough for the wells to be abandoned and for the platform to reach harbour for repairs [32].

5.3. Commercial Boilers

Following the failure by through-thickness cracking of two boilers, a major programme of re-inspection has been going on throughout the United Kingdom over a number of years. The boilers are horizontal multi-tubular shell boilers and are used for a variety of purposes. Among the imperfections found, one which occurs generically is lack of penetration or fusion in the joint between the tube plate and the shell. Tube plates range in thickness from 8-36mm and shells from 10-27mm. The Standard to which the boilers were made, BS 2790, permits no planar imperfections. However, since large numbers of boilers are involved, it would be extremely costly to apply this fabrication specification to imperfections found as a result of in-service inspection. It was therefore decided to carry out an ECA using the PD 6493 procedures. It transpired that fatigue was the ruling criterion. So far 18 boilers have been assessed by the Welding Institute in 13 studies. The analyses have been used to determine the intervals between future in-service inspections. In most cases it has been shown that the normal interval of five years was satisfactory. However, in one case this was reduced to two years, because significant fatigue crack growth was predicted. In another case a change in the operating conditions was recommended. This was that the pressure should be maintained over the weekend. Before the analysis was performed, this boiler had been completely shut down each weekend, so that every week a damaging full pressure cycle was accumulated.

An idea of the economic significance of these assessments is that repair might typically have cost £10,000 per boiler. The total cost of carrying out the ECA would be of the order of £1,000 per boiler.

5.4. Flight Simulator

In flight simulators, an articulated framework and a series of hydraulic actuators, subject the "cockpit" or "flight deck" to accelerations and decelerations, such as would be experienced during the aircraft's manoeuvres. The motion system is subjected to fluctuating loads during simulated flight and under these circumstances the possibility of fatigue failure has to be considered. The Welding Institute was asked to advise on the safe operation of a particular simulator. Fracture mechanics analysis was used to determine the intervals between in-service inspections which would ensure safety. The criterion was that magnetic particle inspection (MPI) should be capable of detecting any crack at one inspection which could grow to a critical size before the next. It was assumed that MPI will reliably detect cracks which are 1mm or more deep. The critical size was assumed to be 3mm (half plate thickness). The simulator question had already been in service for some 15 years and, since it had just been inspected, it could be assumed that no cracks had grown to depth of 1mm in that period. These facts were used in a non-dimension analysis, as proposed by Grover and Egan [33], to show that cracks would not grow to 3mm in under three years. A conservative inspection interval of one year was therefore selected. The dimensionless analysis has
assume an initial flaw size at the start of life. It is interesting that
the smaller the assumed initial size, the shorter the safe interval be-
fore the next inspection. In this case an extremely small slag intrusion
depth at the fillet weld toe of 0.05mm was assumed. A more realistic (and
in this case more optimistic) depth would be 0.2mm.

6. CONCLUSION

This paper has described the outcome of at least 20 years of development
in methods of assessing the significance of weld imperfections with res-
pect to their effect on fatigue performance. This has led to the publica-
tion of a standardised format within which such imperfections can be
assessed. Whilst the IIW Recommendation has only been issued recently, a
similar approach was published in the United Kingdom in 1980 and had been
in practical use before then. Some examples of successful application are
described. It is hoped that these may give other potential users some
confidence in applying these methods.

A point to emphasise is the practical value of having a basis for assess-
ment which can be agreed upon between purchaser, regulator and fabrica-
tor. Unnecessary repair of innocuous defects can be positively detrimen-
tal to safety and adds at least 10% to the costs of welded fabrication.
However, such direct costs can pale into insignificance when compared
with those arising from downtime or late delivery.

7. ACKNOWLEDGMENTS

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Mr. B. Hansen, who has had the unenviable task, as current Chairman of
Sub-Commission VF, of marrying the disparate contributions from the four
Commissions into a viable whole.

8. REFERENCES

[1] Grant, I.M. and Rogerson, J.H. "The importance of contractual re-
quirements in determining the quality costs in the fabrication in-
dustry", Conf on 'Fitness-for-Purpose Validation of Welded Construc-

vessel main seams", Conf on 'Pressure Vessel Standards: The Impact

A study of tolerances for welded ship structures", Conf on 'Fitness
for Purpose Validation of Welded Constructions', The Welding Insti-

[4] Sandor, L.W. "A perspective on weld discontinuities and their accep-
tance standards in the US maritime industry". Ibid.

[5] BS 5500:1986 "Specification for unfired fusion welded pressure ves-
sels", The British Standards Institution, London.


Stress range, $S$ for $2 \times 10^6$ cycles

<table>
<thead>
<tr>
<th>Quality Category</th>
<th>Equivalent Class in IIW fatigue design rules</th>
<th>Stress range, $S$ for $2 \times 10^6$ cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Class in IIW fatigue design rules</td>
<td>in steel N/mm$^2$</td>
</tr>
<tr>
<td>Q100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Q 90</td>
<td>90</td>
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</tr>
<tr>
<td>Q 20</td>
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<td>20</td>
</tr>
</tbody>
</table>

Table I. Values of $S$ for quality categories.

<table>
<thead>
<tr>
<th>Quality Category</th>
<th>Maximum length of slag inclusion*, mm</th>
<th>Limits for porosity expressed as % of area on radiograph</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>As-welded</td>
<td>Stress relieved steel (PWHT)</td>
</tr>
<tr>
<td>Q100</td>
<td>1·5</td>
<td>7·5</td>
</tr>
<tr>
<td>Q 90</td>
<td>2·5</td>
<td>19</td>
</tr>
<tr>
<td>Q 80</td>
<td>4</td>
<td>58</td>
</tr>
<tr>
<td>Q 71</td>
<td>10</td>
<td>No maximum</td>
</tr>
<tr>
<td>Q 63</td>
<td>35</td>
<td>No maximum</td>
</tr>
<tr>
<td>Q 56 to Q20</td>
<td>No maximum</td>
<td>No maximum</td>
</tr>
</tbody>
</table>

* Tungsten inclusions in aluminium alloy welds do not affect fatigue behaviour and need not be considered as flaws under this heading.

Table II. Limits for volumetric flaws in steel and aluminium alloy weldments.
Fig. 1 Section through specimen containing 'gross defects'.

Fig. 2 S-N diagram for pipe butt welds containing 'gross defects' (see Fig. 1).
Fig. 3 Grid of S-N curves for notch classes and quality categories.

Fig. 4 Crack growth rate, da/dN versus ΔK (log/log plot).
Fig. 5 Assessment of weld toe flaws in axially loaded plates with fillet welded attachments:
(a) Relation between actual flaw dimensions and effective flaw parameter;  
(b) Effective flaw size versus plate thickness.
Fig. 6 Typical shop and site welds in a large tubular joint for an offshore platform.
FATIGUE CRACK GROWTH ASPECTS IN STRUCTURAL DESIGN

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Summary

A fatigue crack may initiate quickly and propagate in a metal structure subject to heavy loads as a result of inherent imperfections in the welding operation. This paper reviews current knowledge in this field for better structural design.

1. Introduction

Weld fatigue strength is a function of not only base and filler metal properties, but also of any geometric surface defects or internal defects present (porosity, lack of penetration). Many experiments have shown that the fatigue crack initiation phase from such defects can be reduced and that, as a result, a large part of welded assembly fatigue life consists of propagation.

When designing assemblies, crack propagation laws obtained using fracture mechanics in order to estimate weld fatigue life by calculating the number of cycles needed to make a crack propagate from such defects until it breaks.

The following principle is used to calculate propagation life: the fatigue crack growth rate \( \frac{da}{dN} \) is expressed simply in accordance with the amplitude of the stress intensity factor of part \( K \) using a Paris type law:

\[
d\frac{da}{dN} = C \Delta K^m
\]

where \( \Delta K = \Delta \sigma \sqrt{a} f(a) \)

\( \Delta \sigma \) nominal stress range applied to the structure
\( f(a) \) correction stress factor taking into account structural geometry and loading conditions

Overall weld fatigue life is determined by integrating this law:

\[
N_p = \int_{a_i}^{a_f} \frac{da}{C \Delta K^m} = \frac{1}{C (\Delta \sigma)^m} \int_{a_i}^{a_f} \frac{da}{[\sqrt{a} f(a)]^m}
\]
N_p is the number of propagation cycles of a weld with an initial defect, \(a_i\), at which failure occurs for crack length \(a_f\). For a given weld, if a constant \(a_i\) and \(a_f\) are used, the integral is a constant whatever the constant amplitude stress applied.

To apply fracture mechanics to welds, both the propagation law of the material and the \(\Delta K\) equation corresponding to the assembly geometry must be known.

This paper will review the fatigue crack behaviour of structural steels and welded assemblies. Its purpose is to provide basic elements for the design and in-service monitoring of welded structures and components.

2. Structural Steel Fatigue Crack Strength

Figure 1 shows the various crack regimes [1] and specifies the parameters affecting each one. Particular attention will be paid to regime A, corresponding to low rates and limited by the nonpropagation threshold \(\Delta K_{th}\), and regime B in which the Paris equation (1) is applicable.

2.1 Low Crack Rate Regime

Empirically, the regime A crack curve can be defined by the following equation:

\[
d\alpha/dN = C (\Delta K - \Delta K_{th})^n
\]  \hspace{1cm} (3)

Several parameters affect the level of \(\Delta K_{th}\).

2.1.1 Effect of Stress Ratio (R): For structural steels, Gurney [2] proposes the equation:

\[
\Delta K_{th} = 7.6 - 5.4 R
\]  \hspace{1cm} (4)

Bardal [3] grouped results in Figure 2; the lower limit of the dispersion band is expressed by the equation:

\[
\Delta K_{th} = 6 - 5R
\]  \hspace{1cm} (5)

2.1.2 Effect of Grain Size: Generally speaking, \(\Delta K_{th}\) increases with grain size, as shown by equation (5) proposed by Beevers [5] for ferritic steels:

\[
\Delta K_{th} = 3.8 + 0.0014 d^{1/2} \quad (d \text{ in } \mu\text{m})
\]  \hspace{1cm} (6)
2.2 Medium Rate Regime

In Table 1, Berge [5] has combined the parameters of eq. 1 for a C-Mn structural steel.

In addition to eq. 1 and eq. 2, Austen [6] proposed the more general eq. 7 to model all three regimes, in particular the critical value, $K_C$.

$$\frac{da}{dN} = A \cdot \Delta K^m \left[ \left( \Delta K - \Delta K_{th} \right) \cdot \Delta K \cdot K_C \right]^p \left[ K_C - \frac{K}{(1-R)} \right]^p \quad (7)$$

where $A$, $n$ and $p$ can be obtained by [7]:

$$A = \frac{1}{\left( 4\pi \cdot \sigma_y \cdot E \right)}; \quad n = 2; \quad p = 0.25.$$

2.3 Crack Closure

2.3.1 Phenomenon: When a fatigue crack propagates in an environment loaded in the elasticity range, a plasticized area is created at its tip that induces a field of residual stress. This stress field, in compression at the crack tip in the case of repeated or undulated loads, leads to partial closure of the crack during part of the loading cycle.

This phenomenon, demonstrated by Elber [8], led to the concept of effective stress intensity factor, $K_{eff} = K_{max} - K_{open}$, where $K_{max}$ is the maximum value of $K$ during the cycle and $K_{open}$ is the level of $K$ at which the crack is completely open. In the case of an E36 type structural steel, Figure 3 shows how the significant effect of $R$ on the crack growth rate can be eliminated when $\Delta K_{eff}$ is used [9].

2.3.2 Mechanisms: Many mechanisms can be used to take account of the crack closure phenomenon. Figure 4 shows that for structural steels [10]. In particular, the presence of oxides, whose thickness increases as $\Delta K$ decreases, was examined by Benoit et al. [11] in the case of an AFNOR E36 steel. It is the basis for the beach marking method enabling crack propagation to be monitored [12] through the plate thickness.

2.4 Short Cracks

Application of eq. 1 is restricted to cracks long enough to ensure that the load regime at the root of the notch is stable. Below a certain size that is a function of the microstructure, crack closure occurs: the shorter the crack, the lower the closure. This phenomenon results in a marked decrease of $\Delta K_{th}$ with the length of the crack (Figure 5) [13].
2.5 Biaxial Stress

In an infinite plate subjected to a biaxial stress field, the stress intensity factors in mode I and II for a crack propagating at an angle in relation to \( \sigma_1 \) (maximum principal stress) are:

\[
K_I = \frac{1}{2} \sigma_1 \sqrt{\pi a} \left[ 1 + \frac{\sigma_2}{\sigma_1} \right] - \left[ 1 - \frac{\sigma_2}{\sigma_1} \right] \cos 2\alpha
\]

(8)

\[
K_{II} = \frac{1}{2} \sigma_1 \sqrt{\pi a} \left[ 1 - \frac{\sigma_2}{\sigma_1} \right] \sin 2\alpha
\]

(9)

2.5.1 Proportional Loading

Crack propagation in mode I is obtained either with a crack parallel to a main stress whatever the biaxiality ratio, or in the equibiaxial case whatever the direction of the crack. In the first case, as long as \( \sigma_x/\sigma_y < 1 \) (\( \sigma_x \): stress parallel to the crack; \( \sigma_y \): stress perpendicular to the crack), the propagation path and the cracking law are not modified (Figure 6). When \( \sigma_x/\sigma_y > 1 \), the direction of propagation deviates, which tends to place the crack perpendicular to the maximum stress [14].

In the mixed mode (\( K_{II} = 0 \)), the crack forks. The angle can be predicted using analytical structures [15]; this forking causes the crack to propagate in mode I (\( K_{II} = 0 \)).

2.5.2 Nonproportional Loading

Crack growth rate and path during multimodal loading can be accurately predicted by applying the criterion \( (da/dN)_{max} \) where \( k_I \) is defined on the branched crack (Figure 7) [16].

2.6 Environment

2.6.1 Gaseous Environment

Bignonnet et al. [17] studied the effects of gaseous environments (vacuum, moist air, nitrogen) and frequency for an AFNOR E460 type steel. Figure 8 summarizes the effects observed, in particular:

- increased crack rates in regime B caused by the reduction of surface energy induced by adsorption, which is a function of R and frequency in moist air;
- cracking facilitated by hydrogen at low growth rates, a function of R and frequency, resulting in accelerated cracking and reduced \( \Delta K_{th} \);
- increased crack closure close to the threshold caused by thicker oxides layer; this increase is a function of R, H_2O and O_2 partial pressure, and frequency.

The effect of frequency is linked to its effect on oxides production by fretting-corrosion.
2.6.2 Seawater Environment [18]

- Fatigue crack growth rate are markedly disrupted by seawater in a free corrosion situation. When the $\Delta K$ range is 15 to 40 MPa√m, the growth rate increases (by a factor of 2 to 5) as loading frequency decreases and temperature increases.

Load frequency has been shown to be a main parameter [19]. At frequencies lower than 1 Hz, the crack growth rate increases as frequency declines (Figure 9). It should be noted, however, that when stress intensity levels are lower than 10 to 15 MPa√m, frequency and even the presence of seawater have little effect.

- With cathodic protection at levels used on offshore structures (−800 mV to −1200 mV/ECS), the cracking curve $da/dN = f(\Delta K)$ presents a growth rate plateau that increases as frequency decreases and $R$ and the temperature increase. In this region, the crack growth rate is 10 times greater than in the air. When $\Delta K$ is low, the marine environment has little effect.

The influence of several parameters was studied:

- Scott [20] showed that when the temperature increases from 5 to 20°C and oxygen content from 1 to 7 mg/L, the crack growth rate at 0.1 Hz increases by a factor of 2 in the 15 to 40 MPa√m range;
- reducing seawater oxygen content from 7 mg/L (saturated with air) to 1 mg/L lowers the crack growth rate under free corrosion by a factor of 4 [21];
- in seawater saturated with H₂S, the crack growth rate is accelerated by a factor of 9 to 570 in comparison to air, at $\Delta K = 30$ MPa√m ($R = 0.05$) and at $\Delta K = 12.5$ MPa√m ($R = 0.7$) [22].

All these results suggest that hydrogen embrittlement plays an important role in the cracking process in the marine environment. Cathodic protection encourages absorption of hydrogen, which is likely to diffuse in the metal. Cyclic plastic deformation at the crack tip will permit rapid transportation of the atomic hydrogen in the plastic zone. It can be assumed that there will be competition between the crack advance per cycle due to the mechanical damage and the incubation time needed for hydrogen embrittlement of the volume of metal in which the crack advance will occur.

- The threshold region was studied by several authors. The results of $\Delta K_{th}$ are given in Table 2. They indicate that $\Delta K_{th}$ is generally higher under cathodic protection than under free corrosion and even in the air. This can be attributed to the effect of calcareous deposits inside the crack. Precipitation and adhesion of these cathodic protection products are primarily a function of crack tip
potential, current density, seawater circulation speed, temperature, pH inside the crack and, indirectly, crack geometry (length and shape).

2.6.3 Temperature

A recent bibliographic review [26] specified the effects of temperature and R on the level of ΔKth:
- for low R values, ΔKth passes falls between 100 and 200°C minimum; the levels of ΔKth between 300 and 400°C are close to those observed in the ambient environment;
- this trend decreases as R increases; the effect of temperature is not significant for a high R (> 0.8).
This phenomenon can be explained in terms of crack tip closure, caused by roughness and the presence of oxides.

2.7 Variable Amplitude Loading

2.7.1 Effect of Overloads

The significant effect that overloads had on the cracking of an AFNOR E355 steel was shown by Chehimi et al. [27], who demonstrated that crack blocking can be obtained when the overload rate reaches a certain value or when the number of overloads is sufficient for a given overload rate (Figure 11). The authors link this behaviour to the development of the cyclic behaviour of the material at the root of the notch.

2.7.2 Loading Spectra

Under variable amplitude Gauss type load combinations representative of successive sea conditions, several authors [28] to [32] have described propagation as a function of an "equivalent" load based on long-term distribution of stress amplitudes, which does not take into account the effects of cycle interaction. Thus, if da/dN = C ΔKm, ΔK = f(a/w) ΔP, and ni is the number of times where the stress ΔPi is applied, the crack length increase is C ΔPi

For a sequence of N cycles, an equivalent load can be calculated that will give the same result as variable amplitude loading:

\[ ΔP_{eq} = \left[ \frac{1}{N} \sum_{i=1}^{N} ΔP_i \right]^{1/m} \] (10)
This is the $m$th root of the statistical moment $m$ of the stress amplitude distribution. In the specific case where $m = 2$, this is the root-mean-square value, or effective value, of the sequence. The propagation law described as a function of $\Delta K_{eq}$ (Figure 12) is similar to that obtained at constant amplitude [28].

Fatigue crack propagation prediction based on this approach is acceptable provided that the average crack growth rate is not too fast. When the crack lengthens rapidly, i.e. at more than $5 \times 10^{-8}$ m/c, the assumptions permitting this type of approach are no longer respected. Short-term variations of the function of probability density then dominate and the concept of average growth rate loses its meaning [28].

The variable amplitude loading approach using the concept of equivalent stress can be applied structures where cracks propagate slowly on average and where factor $K$ evolutions in accordance with crack length are very slow.

3. Fatigue Cracking in Welded Assemblies

The literature deals with cracking specimens from welds (3.1) or from welded assemblies (3.2).

3.1 Weld Specimens

3.1.1 Role of Various Parameters

- Base Metal and HAZ Comparison

Most results published concern fracture mechanics specimens with a mechanical notch running through the plate thickness and located in the HAZ. At the same $\Delta K$ level, a lower crack growth rate ($da/dN$) is generally observed in the HAZ than in the base metal (BM). However, the difference between these rates decreases as $\Delta K$ increases [33] to [35]. This reduction in rate is generally accompanied by a deviation of the crack from its initial plane to the base metal. Some authors [34] [36] have explained this deviation by the difference in mechanical properties between the HAZ and the BM, with the crack heading from a harder to a softer microstructure. Most mention the effect of a residual biaxial stress field.

- Specimen Sampling Direction

Specimen sampling is performed so as to make the crack propagate either parallel or perpendicular to the weld, or in the direction of the thickness of the assembly [37] [38].
Traversing Crack

i - Sampling parallel to the weld: In the case of a specimen with a lateral notch (CT type), Lieurade et al. [37] and Kitsunai et al. [39] showed that, where \( R \approx 0 \), there was a very significant decrease in crack growth rate in relation to that of the base metal (Figure 13). On the other hand, using a specimen with a central notch (CCT type) resulted in, for \( R = 0 \), growth rates higher than those of the base metal [40] [41] (Figure 14).

ii - Sampling perpendicular to the weld: Makhenko et al. [42], Trufiakov et al. [43] and Ignat'eva et al. [44] showed that propagation kinetics vary with each case. If the mechanical notch starts at the weld bead, accelerated propagation is observed, while if it is located between two beads, crack growth is slowed down considerably. This was confirmed by Glinka [40]. In the case of a specimen with a lateral notch (CT specimen), the behaviour of a crack propagating toward the weld bead depends on the distance from the crack tip to the weld bead [39]. The greater this distance, the slower the propagation.

Propagation through the Thickness

In the case of multipass welding of 40 or 75 mm thick plates, Lieurade et al. [37] observed a crack rate in the thickness where \( R = 0.1 \) comparable to that in base metal where \( R = 0.7 \) (Figure 13).

Residual Stress

All the above result from the residual welding stress field in the specimens. Accelerated cracking is linked to residual traction stress perpendicular to the crack, while slower cracking corresponds to compressive stress.

- Plate Thickness

Lieurade et al. [37] demonstrated the significant effect of thickness (20 or 40 mm) on cracking rates in the HAZ. Where \( R = 0 \), they observed a reduction in crack rate that increased with thickness. They proved that this reduction is caused by an increase in residual compressive stress in the center of the specimen.

- \( \Delta K \) Level

In the case where slowing down of the crack growth rate is observed in the HAZ, the difference in crack growth rate between the HAZ and BM decreases then is eliminated when \( \Delta K \) increases (Figure 13). Conversely, the difference is greatest at the threshold level ( \( \Delta K_{th} \) ) [45] [46].
- **Base Metal Yield Strength**

Under the same sampling (CCT specimens) and loading (R = 0) conditions, Ohta [41] obtained comparable results for steel assemblies with yield strengths ranging between 300 and 550 MPa. However, Lieurade et al. [37] observed a slowdown in crack growth rate in the HAZ that increased as base metal yield strength increased (280 < \( \sigma_y \) < 515 MPa) (CT specimen, R = 0). A higher yield strength results in a higher level of residual compressive stress in the center of the specimen.

These authors confirm the relationship between \( m \) and \( \log C \) shown by Gurney [47] for structural steels (BM), including the results obtained for molten metal (MM) and the HAZ (Figure 15a). Given the relationship between \( m \) and \( C \), these authors examined the evolution of \( m \) as a function of \( \sigma_y \). The results obtained generally on thin specimens fell within the same dispersion band, corresponding to a variation of \( m \) close to ± 15% for a given \( \sigma_y \) value (Figure 15b). Moreover, the evolution of \( m \) is relatively low at yield strengths corresponding to structural steels.

3.1.2 Crack Closure

Figure 16 shows application of this concept to cracking in the HAZ of an E36-Z steel welded joint [37]. The results for R = 0.1 expressed as a function of \( \Delta K_{eff} \) are superposing on those obtained for the base metal with R = 0.7 (R values at which the crack is completely open).

Several authors [39] [48] [49] have measured crack opening during tests on specimens from welded assemblies, and have expressed the crack rate as a function of \( \Delta K_{eff} \). In particular, Fukuda et al. [49] applied the concept of \( \Delta K_{eff} \) in crack tests with R ratios between −∞ (repeated compression) and + 0.5. The results vary widely with R (Figure 17a) when expressed as a function of \( \Delta K \), while they are similar when they expressed as a function of \( \Delta K_{eff} \) (Figure 17b).

In the field of very low crack rates (\( da/dN < 10^6 \text{mm/cycle} \)) and the threshold \( \Delta K_{th} \), the base metal and welded assembly results, expressed as a function of \( \Delta K_{eff} \), are located within the same dispersion band [50].

In fact, whatever the loading conditions (R and \( \Delta K \) levels) and residual welding stress (welding process, yield strength, thickness, specimen sampling method), a single curve is obtained when studying only that portion of the cycle during which the crack is completely open.

This conclusion indicates that the law for crack growth rate in a welded assembly is primarily dependent on the evolution of residual stresses at the crack tip. This explains the often contradictory results found in the literature.
3.2 Welded Joint

3.2.1 Crack Development

Figure 18 shows crack evolution in accordance with the number of cycles for a T joint [51] [52]. Some microcracks existing or forming along the weld toe coalesce to form small semi-elliptical cracks with high $a/2c$. When these microcracks grow, $a/2c$ decreases (Figure 19). During the next phase, crack analysis is complicated by closure (short crack) and interaction effects. This phase ends when a group of macrocracks coalesce to form a semi-elliptical crack (low $a/2c$).

It should be noted that these initial phases, which only correspond to a crack depth of around 0.2 $T$ ($T$ equals weld joint thickness), represent the largest part of the life of an welded joint.

3.2.2 Short Cracks

Given its importance, the short crack phase was studied using a cruciform welded joint. Verremann et al. [48] [53] demonstrated both experimentally and theoretically that the initially rapid growth of the crack slowed down in accordance with crack closure conditions that developed when the short crack became longer (Figure 20).

3.2.3 Crack Monitoring

Various methods have been used to monitor cracking in welded joints, in particular:
- ink [54] or beach marking [54] [12] [55];
- the potential drop method [51] [56];
- the compliance differential measurement method using electric gauges [57].

The results obtained have enabled theoretical calculations on crack growth to be validated and fatigue life to be predicted.
- T joints [58] [54];
- pipe welds [55] [59].

3.3.4 Effect of Thickness

The drop in welded joint fatigue strength that accompanies increased plate thickness [60] can be described using a fracture mechanics model. For this, it is assumed that assemblies of the same type but different thicknesses are geometrically similar and that initial cracking conditions are independent of thickness ($a_i$ = constant). Figure 21 shows the effect of thickness; the higher stress gradient for thinner joints results in a lower $\Delta K_i$, thus a lower initial cracking rate.
3.2.5 Effect of Environment

The literature primarily discusses the work in a marine environment (seawater, low frequency) [18]. It shows that behaviour differs under free corrosion or cathodic protection conditions [61] (Figure 22).

- **Free corrosion:** The increased growth crack rate under free corrosion conditions as opposed to air can be explained by the rapid appearance of edge cracks (multiple initiation) that accelerate cracking and reduce the dispersion of results.

- **Cathodic protection:** Few cracks are initiated; these cracks retain their semi-elliptical shape for a long time which, with the same a value, corresponds to a lower \( \Delta K \) than for an edge crack. This explains the difference in behaviour. In addition, formation of calcareous deposits - which increases as potential becomes more negative - reduces propagation by partial closure of short cracks.

A contrasting effect has been observed in the case of long cracks or high \( \Delta K \): crack growth rates increases when either frequency or potential decreases. The adverse effect of too negative a potential is associated with localized plastic deformation encouraging hydrogen embrittlement.

In the case of cathodic protection, deeps notches have a significant effect; localized hydrogen embrittlement may accelerate crack initiation and initial cracking phase normal to these notches.

3.2.6 Improvement Techniques

Improvement techniques modify weld surface properties. They only affect the initiation phase and the early crack stage, which generally represents the largest part of the assembly fatigue life.

Improved weld bead geometry (increased connection angle and radius) results in markedly increased propagation lives (Figure 23) [62].

Shot peening [57] and hammering [63] introduces residual compressive stress that delays crack initiation and considerably slows down the first propagation stage (Figure 24).
4. Welded Structure Fatigue Calculation

4.1 Estimating the Stress Intensity Factor ($K_I$)

Many methods are used to calculate $K_I$. They are applicable to semi-elliptical or edge cracks. Generally, the cracked component is modelled using the finite element method. $K_I$ is expressed as:

$$K_I = M \cdot \sigma \cdot (\pi a)^{1/2}$$  \hspace{0.5cm} (10)

where $\sigma$ is the nominal stress and $M$ is a correction factor.

For a welded assembly, $K_I$ becomes:

$$K_I = M_K \sigma (\pi a)^{1/2}$$  \hspace{0.5cm} (11)

For very small cracks, $M_K$ is usually equal to the coefficient of elastic stress, $K_t$. In the general case where a membrane stress, $\sigma_m$, and bending stress, $\sigma_b$, exist simultaneously, the following equation is applicable:

$$K_I = M_{km} \sigma_m (\pi a)^{1/2} + M_{kb} \sigma_b (\pi a)^{1/2}$$  \hspace{0.5cm} (12)

Many authors have specified the $K_I$ equation for most weld configurations (load type and mode). Recent syntheses have been published [64] [65]. Only the approach will be reviewed here; several examples will be given.

4.1.1 Weld Toe Cracking

- Fillet welds

To calculate $K$ for a crack initiated at the weld toe and propagating through the plate thickness, Maddox [66] assumed a semi-elliptical surface defect of the $a$ and $c$ semiaxes (Figure 25). The stress intensity factor, obtained by a finite element analysis, is:

$$K = \sigma \sqrt{a} \cdot \frac{M_s + M_t + M_k}{\phi_o}$$  \hspace{0.5cm} (13)

where $M_s$ = free surface correction
$M_t$ = finished thickness correction
$M_k$ = correction for stress concentration
$\phi_o$ = stress concentration factor

$$\phi_o = \int_0^{\pi/2} \left[ 1 - \left( 1 - \frac{a^2}{c^2} \right) \sin^2 \varphi \right]^{1/2} d\varphi$$
K was calculated for a cruciform joint and an elongated
defect (x = 1) in accordance with a/B (B = plate
thickness) and for θ variable connection angles.

- θ = 0, \( K = \sigma \sqrt{\pi a} \left[ 1.22 - 0.231 \frac{a}{B} + 10.55 \left( \frac{a}{B} \right)^2 - 21.7 \left( \frac{a}{B} \right)^3 \right] + 33.19 \left( \frac{a}{B} \right)^4 \)

- θ ≠ 0, there is stress concentration at weld toe and K is
higher in the area of a/B = 0 (Figure 26).

The effect of weld bead geometry is taken into account
using the preceding models [66] to [69]:
- connection angle and radius [63]
- weld side
- scale effect [70]
- stiffener thickness [71].

The same models can be used to specify the effect of
initial defect size and shape (a = f(c)).

The role of poor plate alignment or joint misalignment can
be described using finite element analysis [73], by
superposition of traction (Δσ_A) and bending (Δσ_B) load
modes.

- Butt welds

The Maddox model [67] described above is applicable to butt
welds. Gurney [68] adapted it to finite element
calculation. Lawrence’s model [73] is more elaborate; it
is applicable to perfectly symmetrical butt welds.
Lawrence used finite element calculation to determine the
stress field along the cracking plane for different weld
bead geometries (θ, θ) (Figure 27). He adjusted the
\( \frac{\sigma_x}{\sigma_{nom}} \) curves thus obtained using a four-stage polynomial
function:

\[
\frac{\sigma_x}{\sigma_{nom}} = \sum_{i=0}^{4} b_i \left( \frac{x}{B} \right)^i
\]

and gave the chart of the constants b_i in accordance with
the geometric parameters θ and θ (or h and W). He used the
following equation for the stress intensity factor K
corresponding to a crack in a semi-infinite environment
subjected to a nonuniform load \( \sigma_x \):

\[
K = 1.1.\sigma_x.\sqrt{\pi a}.\int_{0}^{a} f(x/a).\left( \frac{d\sigma_x}{da} \right)dx
\]

where \( f(x/a) = 0.8 \left( \frac{x}{a} \right) + 0.04 \left( \frac{x}{a} \right)^2 + 3.62.10^{-5}.\exp \left[ 11.18 \left( \frac{x}{a} \right) \right] \)
As in the case of fillet welds, the models proposed take into account the weld bead geometric parameters [73] and axiality defects [71].

Analysis of the behaviour of transversally butt welded plates of differing thickness was taken into account [71] by superposing in the axial loading mode a bending moment caused by the gap between the median planes of the plates.

**Pipe Welds**

In the case of pipe node cracking at the chord-brace connection, a three-dimensional approach can theoretically be used; however, such an approach poses numeric problems and there is a lack of specific knowledge regarding certain cracking mechanisms. This is why simplified approaches were used.

. **Surface Cracking:** Tompkins et al. [74] analyzed surface cracking around the node intersection using the following integral:

\[ K = \frac{2\sqrt{c}}{\sqrt{\pi}} \int_{0}^{c} \frac{\sigma d\chi}{\sqrt{c^2 - \chi^2}} \]  \hspace{1cm} (16)

where \( \sigma = f(\chi) \) takes into account the evolution of the initial stress concentration factor along the chord-brace intersection. This analysis does not take into account stress redistribution during cracking. The stress intensity factor can be expressed as follows:

\[ K = Y_{\sigma} \cdot Y_{s} \cdot \sigma_{R} \sqrt{\pi a} \]

where \( Y_{\sigma} \) corresponds to the evolution of the stress field \( Y_{s} \) takes into account geometric effects \( \sigma_{R} \) is the radial stress at the hot spot.

. **Trough cracking:** Van Delft et al. [75] proposes a model that shows trough cracking of T tubular joints. For this, they modified the formulae applicable to semi-elliptical defects present in infinite plates. They used the following equation:

\[ K_{I} = (M_{m} \cdot \sigma_{m} + M_{b} \cdot \sigma_{b}) \frac{1}{E_{K}} \sqrt{\pi a} \]  \hspace{1cm} (17)

where \( a \) is crack depth, \( E_{K} \) an elliptical integral in the form:

\[ E_{K} = [1 + 1.47 \left( \frac{a}{c} \right)^{1.64}]^{0.5} \]

The correction factors \( M_{m} \) and \( M_{b} \) depend on \( a/c \) and relative depth \( a/t \); \( \sigma_{m}/\sigma_{b} = 3 \).
To depict the first phase of cracking and the "short crack" effect, the authors used the crack depth correction factors, $l_0$, suggested by El Haddad and Topper [76], written:

$$l_0 = \frac{1}{n} \frac{K_{th}^2}{\sigma_e}$$

where $\sigma_e$ is the material fatigue limit.

4.1.2 Cracking from Weld Defects

- Lack of Penetration

. Butt weld: Calculations are performed using a specimen equivalent to the joint studied, i.e. in this case an infinite plate with a thickness of $2W$ and a central crack with a length $2a$ (Figure 28a).

Harrison [77] expressed $K$ as follows:

$$K = \sigma \sqrt{a} \cdot [(W/Wa) \cdot \tan(\pi a/W)]^{1/2}$$

(18)

For ground butt welds, Lawrence and Munse [78] used:

$$K = \sigma \sqrt{a} \cdot \cos(\pi a/W)^{1/2}$$

(19)

Their results were very similar to those obtained experimentally (Figure 28b).

. Cruciform Joints: Frank [79] calculated a $K$ solution specifically adapted to a cruciform weld root using finite element techniques. This solution was used by Maddox [80] in the following form:

$$K = \frac{\sigma_p}{1 + 2H/T_p} \left[ A_1 + A_2 \left( \frac{a}{W} \right) \right] \left[ \frac{a}{a} \sec(\pi a/W/W) \right]^{1/2}$$

where $\sigma_p$, $H$, $T_p$, $a$, $w$ are defined on Figure 29; $A_1$ and $A_2$ are polynomials of $H/T_p$.

Using this definition of $K$, integration of the crack growth rate law (with $m = 3$) results in the following equation:

$$I = C W^{1/2} \left( \sigma_p \right)^3 N$$

where $I$ is an integral whose evolution in accordance with $a_1/W$, for given values of $H/T_p$, is specified in Figure 29.

- Porosity

Various authors have examined the effect of porosity by assimilating it as circular [81], elliptical [82] or ellipsoid [83] plan defects.
4.2 Taking Various Parameters into Account

4.2.1 Evolution of Crack Geometry

To take into account the specific development of cracks in the first stages of propagation (see 3.2.1, 3.2.2), various models have been proposed.

Vosikovsky et al. [84] developed a "multiple crack" model taking into account crack coalescence.

In Figure 30, it is assumed that the edge crack is reached when \( a/2c = 0.1 \). This figure shows that models with single or multiple cracks are comparable when \( a/T \) is very low, while the multiple crack model only shows the later cracking stages.

4.2.2 Weld Microgeometry Scatter

To take into account microgeometry scatter at the weld toe (connection angle and radius, initial defect, \( a_i \)) and cracking growth rate law parameters, Engesvik and Moan [85] used a probabilistic approach. The statistical distribution of parameters measured was determined.

A Monte Carlo simulation was used to derive the fatigue cracking life uncertainty associated with these parameters.

4.2.3 Residual Stress Field

To quantify the effect of residual welding stress on fatigue cracking, it is necessary to evaluate the stress field and to determine the stress intensity factor, \( K_R \), induced at the crack tip by residual stress.

- **Residual stress evaluation:** The initial stress field can be evaluated by finite element calculations or by experimental measurements such as X-ray diffraction.

- **\( K_R \) determination:** Parker [86] reviewed the various methods available for evaluating the stress intensity factor, \( K_R \), induced at the crack tip by residual stress; all these methods require knowledge of the profile of residual stresses present in the uncracked specimen and smoothing of this profile using a polynomial. These methods include the Green functions and the Bückner method.

Figure 31 presents an application of these methods to an initial residual welding stress field [87]. In this case, the evolution of \( K_R \) with the length of the crack was determined using a Green function. The \( K_R \) remains positive even for initial levels of \( \sigma_R \) in compression.

In the case of cracking perpendicular to a butt weld, Makhenko and Trufyakov et al. give applications of these methods for a relatively short \( (a < 20 \text{ mm}) \) [88] or long [89] crack.
- **Superposition principle:** The most common approach used to show the effect of residual stress on fatigue cracking superposes stress intensity factors corresponding to initial residual stress and applied stress (Figure 32).

- **Application:** Application of eq. (1) to (6) enables the $\Delta K$ and $R$ parameters to be determined. For a crack propagating in a residual stress field, $K_R$ is added to $K_{\text{min}}$ and $K_{\text{max}}$; two cases may occur:
  
  . where $K_{\text{min}} + K_R > 0$, $\Delta K = K_{\text{max}} - K_{\text{min}}$
  
  and $R = (K_{\text{min}} + K_R)/(K_{\text{max}} + K_R)$

. where $K_{\text{min}} + K_R \leq 0$, $\Delta K = K_{\text{max}} + K_R$ and $R = 0$.

This type of approach was successfully used by various authors, in particular Parker [90] (typical welding stress fields) and Glinka [40] [91] (complex stress fields introduced by different welding conditions).

### 4.2.4 Variable Amplitude Loading

Calculation of crack growth life under variable amplitude loads can be performed in two different ways:

- **$\Delta K_{eq}$ approach (see 2.7.2):** in this case, the calculation is performed in the same way as for a constant amplitude load (see 1);

- **Application of a cumulation rule:** in this case, the assumption is made that the load distribution consists of successive blocks of $n_i$ cycles corresponding to a stress variation $S_{r,i}$ giving the following equation:

  $$n_i = \frac{1}{C.(S_{r,i})^m} \int \frac{a_i+1}{a_i^{0.5}F} da$$

The crack growth life is the sum of $n_i$ such as $a_i + 1 \leq a_f$.

### 4.3 Design Codes

Several welded structure fatigue design codes or recommendations use an approach based on fracture mechanics concepts.

- **PD 6493 (BSI):** This recommendation [92] contains diagrams enabling the harmfulness of defects to be defined and the corresponding levels of admissible stress to be determined. These diagrams were obtained using fracture mechanics analysis, whose principles applied to plan defects are discussed by Harrison [93].

- **WES 2803 Standard (JWES):** Kanazawa et al. [94] described the use that is made in this code of fracture mechanics concepts with regard to welded structural steel assemblies ($UTS < 100$ kg/mm²).
- DnV Rules on the Construction of Offshore Steel Platforms: These rules [95] include a simplified fatigue design procedure for butt welds and fillet welds. In this procedure, $\Delta K = \Delta \sigma \cdot k \cdot a^n$, where $k$ and $n$ are parameters depending on the geometry of assemblies in the form of charts.

- International Welding Institute: "Fitness for Purpose" [96]

This recommendation, used to evaluate weld defects, uses charts to show the harmfulness of planar defects using fracture mechanics analysis.

Conclusion

The knowledge acquired over the last ten years on metal structure fatigue cracking has meant that models could be developed to accurately describe the phenomena observed. These models, which take early cracking stages into account, enable structure reliability to be improved.

References


[34] Rabbe et al. (1979), IIW Doc XIII - 914-79.


[37] Lieurade, H.P. et al. (1982), IABSE Colloquium Lausanne, 137-144.


[48] Verreman, Y. et al. (1985), The Mechanism of Fracture, Pub. ASM.


[73] Lawrence, F.V. (1973), Weld. J., 212 s - 220 s.
Table 1 - Fatigue crack parameters $C$ and $m$ (Eq.1) for C-Mn structural steels BS 4360 Grade 50 or similar, in air. (from [5])

<table>
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<th>$C (\frac{m}{(MPa\sqrt{m})^m})$</th>
<th>$m$</th>
<th>Validity</th>
<th>Comments</th>
</tr>
</thead>
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<td>$7.1 \times 10^{-12}$</td>
<td>3.0</td>
<td>$R \geq 0.0$</td>
<td>Mean values</td>
</tr>
<tr>
<td>$5.32 \times 10^{-11}$</td>
<td>2.53</td>
<td>$R \geq 0.0$</td>
<td>Upper bound</td>
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<tr>
<td>$5.9 \times 10^{-12}$</td>
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<td>$\Delta K \leq 22 \text{ MPa} \sqrt{m}$</td>
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</tr>
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<td>2.65</td>
<td>$R \geq 0.0$</td>
<td>Mean values</td>
</tr>
<tr>
<td>$3.9 \times 10^{-12}$</td>
<td>3.1</td>
<td>$\Delta K &gt; 22 \text{ MPa} \sqrt{m}$</td>
<td>Mean values</td>
</tr>
<tr>
<td>$5.2 \times 10^{-12}$</td>
<td>3.0</td>
<td>$R = 0$</td>
<td>Mean values</td>
</tr>
<tr>
<td>$9.5 \times 10^{-12}$</td>
<td>3.0</td>
<td>$R = 0$</td>
<td>Upper bound</td>
</tr>
</tbody>
</table>

Table 2 - Threshold stress intensity range for AFNOR E 355 structural steel in sea water (from [18])

<table>
<thead>
<tr>
<th>$R$ ratio</th>
<th>freq. (Hz)</th>
<th>temp. °C</th>
<th>$\Delta K_{th}$ (MPa/√m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Air</td>
<td>free corr.</td>
<td>- 850 mV</td>
</tr>
<tr>
<td>0</td>
<td>1/6</td>
<td>10</td>
<td>6.3</td>
</tr>
<tr>
<td>0.05</td>
<td>10</td>
<td>6.29</td>
<td>8.75</td>
</tr>
<tr>
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<td>3.0</td>
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</tr>
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<td>10</td>
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</tr>
<tr>
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<td>1/6</td>
<td>5.67</td>
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<tr>
<td>0.75</td>
<td>10</td>
<td>3.95</td>
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</table>
Fig. 1 - Sigmoidal variation of fatigue crack growth rate of long cracks as a function of $\Delta K$

Fig. 2 - Effect of stress ratio $R$ on the threshold stress intensity range

Fig. 3 - Effect of $R$ on the crack growth rate as a function of $\Delta K_{\text{eff}}$ (AFNOR 835 steel)

Fig. 4 - Schematic illustration of various mechanisms of fatigue crack closure

Fig. 5 - Variation of threshold stress intensity factor $\Delta K_{t}$ at short crack sizes for different materials

Fig. 6 - Crack growth kinetics for different conditions of biaxiality in comparison with the uniaxial situation
Fig. 1 - Presentation of the \( \frac{\text{d}a}{\text{d}N} \) max criterion

(a) derivation of \( \Delta R(\theta) \) and \( R(\theta) \) for any value of branching angle

(b) determination of \( da \) corresponding to \( \Delta R(\theta) \) and \( R(\theta) \)

\[ dN \]

The locus of couples (\( \Delta R(\theta) \), \( R(\theta) \)) exhibits a maximum in terms of fatigue crack growth rate: the angle \( \theta \) at this point is the branching angle.

**CRACK SURFACE OXIDATION EFFECT**

Fig. 8 - Influence of oxide induced closure on fatigue crack growth characteristics

1 Purely mechanistic, mode I crack propagation behaviour (in vacuum)

2 Effect of closure and mode II on the near-threshold, without environmental effect

3 Hydrogen-assisted crack propagation behaviour

4 Influence of adsorption and H diffusion at low R ratio

5 Crack growth behaviour at low R ratio in moist environment
Fig. 9 - Influence of the load frequency on the fatigue crack growth in sea water, free corrosion

Fig. 10 - Influence of the frequency on the fatigue crack growth in sea water with cathodic protection

Fig. 11 - Variation of the number of cycles of delay with the number of overload peaks. (AFNOR E 36 steel) $K_s = 16.2 \, \text{MPa}/\text{m}$

Fig. 12 - Fatigue crack growth rate under random loading and the use of the equivalent stress
Fig. 13 - Effect of cracking direction on the crack growth rate (APNOR B 36 Steel)

Fig. 14 - Effect of various loading conditions on the crack growth rate of welded joints

Fig. 15 - a and c variation for various steel grades

Fig. 16 - da/dN variation as a function of ΔK or ΔKeff
Fig. 17 - \( \frac{da}{dn} \) vs \( \Delta K \) (L/\( t \)) for longitudinal weld bead

Fig. 18 - Crack development in T plate joints

Fig. 19 - Variation of aspect ratio with crack depth

Fig. 20 - Crack length versus cycle for loading below nominal yield stress
Fig. 21 - A simplified model for describing the effect of thickness in fatigue failures developing from the weld toe.

Fig. 22 - Growth of the first initiating fatigue cracks in 78mm thick joints tested in seawater at free corrosion, optimum cathodic overprotection, and in air.

Fig. 23 - Influence of weld toe radius on crack growth.

Fig. 24 - $da/dN$ vs $a$ in the case of hammer peened specimens with longitudinal welded stiffeners.
Fig. 25 - Details of weld toe crack: 
a) section, 
b) plane of fracture

Fig. 26 - Influence of \( a/W \) and \( \theta \) on \( K \) 
(non-load-carrying fillet weld)

Fig. 27 - Butt joint, geometrical parameters 
according to LAWRENCE

Fig. 28 - a) butt joint ground flush with lack of penetration 
b) calculated crack growth evolution and experimental
Fig. 29 - Crack propagation integral for weld crack in cruciform joint as a function of initial flaw size.

Fig. 30 - Crack aspect ratios, measurements and models, for fatigue cracks at toe of full penetration welds in T-plates.

Fig. 31 - $K_e$ evolution during cracking in a residual stress field due to welding.

Fig. 32 - Superposition method.
Summary

The design of constructions against fatigue due to random loading is not just a matter of estimating the endurance under an estimated load spectrum of the fatigue-critical sections. Much more effects have to be drawn into the design process.

In order to distinguish different design criteria, the loads on the structure are split up in service- and environmental loads and in incidental loads due to incidents as are accidents, but also due to loads from the tails of the statistical load distributions that are not accounted for in a conventional life assessment.

The possible influences of these incidental loads on the endurance is discussed with examples, and it is emphasized that in order to design fail-safe and safe-endurance qualities into a structure a constant fatigue quality is necessary.

1. Introduction

The emphasis in this lecture will not be on experimental achievements or on modelling for sequence effects in random fatigue. It will be on basic design considerations that have to be taken into account in order to realise a construction with a predictable behaviour under random loading.

In order to get the full extend of the influence of random loading on the reliability of structures with regards to fatigue, it is sensible to begin at the beginning.

Structures are loaded by forces that fluctuate in time. When the fluctuations are large or when the stress- or strain concentrations within the structure are high, there is a real danger of failure of that structure by fatigue. In those cases the designer has to estimate the endurance of his structure and has got to access the risks involved when the structure might fail.

The design procedure with regard to strength considerations consists in general of the following parts:

- Preliminary design
- Estimation of external loads during the anticipated life
- Estimation of the response
- Determination of forces in the structural elements
- Design
- Estimation of product quality
- Design of structural details
- Prediction of life and structural reliability.

The repeated use of the word "estimation" is to remind that everything on paper - software we say nowadays - is derived from models generated on the base of past experience.
So experimental verification is often mandatory, if only because:
- models are steadily being improved.
- materials are steadily upgraded.
- production technology is continuously changing.

2. Types of loading

From design point of view it is feasible to distinguish between loads from different origins, fig.1, viz.:
- Service loads
- Environmental loads
- Incidental loads

Just a few simple examples to illuminate the picture:

Service loads are caused by the function of the construction or the machinery.
for a vehicle these are due to steering, braking and acceleration and due to the payload.
for a bridge they are due to the forces from these vehicles.

Service loads are usually to be described by statistical parameters, but their magnitude is often limited or can be controlled.

Environmental loads are due to the influence of the environment on the structure. They usually can be split up in different but interrelated parts, e.g.:
mechanical - corrosion - erosion - etc.
The examples below only regard the mechanical part.
for a vehicle they are mainly due to the road profile
for an offshore structure the loads from wind, waves and currents are the dominant ones.

Environmental loads are for the greater part of random nature (e.g. wind and waves are, tidal streams are not) but in contrary to service loads, their magnitudes cannot be controlled.

To summarize:
Both service loads and environmental loads are usually of random nature. The force spectra from these loads during the lifetime of the structure are the base of endurance calculations and -tests.

In general it is sufficient for random variables to determine the probability density and the spectral density. This holds in principle also for loads L causing fatigue damage. However, the probability density of the load is measured as the relative time spent in a window $L_i < L \leq L_i + dL$.
Per definition time does not enter fatigue-damage relationships and therefore it is also relevant to determine the distribution of parameters, e.g. ranges, that are relevant to the fatigue process. However the manipulations necessary to convert the primary statistical parameters into secondary ones are relatively easy ones.
In handling statistics or random processes there are two aspects that are of importance viz.:

- The stationarity of the random parameters in the time domain.
  Normally the sampling of load parameters is performed during periods that are small as compared to the life of the type of structure involved and so the results are to be extrapolated into the future.

During the life of the structure changes of deterministic or of random nature may occur.
Examples:
In times of economic recession the maintenance of roads tends to lag, and the surface roughness of the roads will increase. This will change the loads on vehicles during their life.
The change in traffic density in the future for a bridge that is designed to last 100 years.

It is clear that to anticipate this type of changes is a difficult and not very rewarding task for a designer.

- The variation of the random parameters across a fleet of similar structures. Formally this problem can hardly be solved. Technically it is solved by sampling load parameters from a number of similar structures used under different conditions.
From these case histories a relevant load spectrum is determined.

Incidental loads (our major subject) find their origin in causes that can hardly be traced by common considerations, whether deterministic or statistic. So they are not predictable in the normal way or if they are predictable the probability of occurrence during the anticipated life of the structure is smaller or much smaller than one (so the tails of the probability densities). Examples of the first type are collisions for vehicles or earthquakes for offshore structures. Examples of the second type are a very large pothole in the road for a vehicle or the 1000-year wave for an off-shore structure.
Furthermore a third type can be distinguished, viz. loads caused by functional failure of a part of the structure itself, e.g. a blown tyre for a vehicle or the fatigue failure of a brace of an offshore jacket structure.

Because incidental loads have a very low chance of occurrence, and because their magnitude can be high and is mostly not predictable, they are not suitable to be incorporated into the ordinary design procedures. Therefore they are to be treated in a deterministic way and that means that a decision is made about the ultimate severity of loading that a structure should withstand and what damage is allowed from these loads.
Two examples:
Offshore platforms are often designed for a 30-year fatigue life together with a capacity to withstand the loads of waves that occurs once in a hundred years on the average.
The dikes in the Netherlands are or are to be so high that a sea-state that occurs once in a thousand years on the average will not cause overflow.

It is very clear that the figures (100-year wave, 1000-year sea-state) are chosen ones, and that ethic and economic backgrounds are to play a dominant role in the determination of these figures.
However, it is also clear that after the 1000-year wave - which may
happen tomorrow or 10,000 years after tomorrow - the Northsea could look very different from now, or that after the 10,000 years sea-state half of the Netherlands would be flooded.

From the above it is clear, that designers are reluctant to design against incidental loads, because there are always loads or cases that are more severe than the ones the structure is designed for, and if these happen society will blame them. Therefore design rules, laid down by national or supra-national committees, are to take over these responsibilities.

3. Structural Response

Starting from the prime loads as are wave height, road profile, relations between vehicle speed and manoeuvre etc. there are a lot of steps to be taken before a life estimate can be made.

3.1 Loads to forces

The primary loads described in par. 2 cause dynamic forces \(^1\) on the structure and the latter will be excited by these forces.

The transfer function between loads and forces can be a linear one, but often is not (e.g. drag, tyre characteristics) and it may be stationary but often it is not because of changes in time of the interface (e.g. marine fouling, wear of tyres) with the environment.

The influence of the dynamics of the structure has to go into this transfer function, because of possible feedback. On the one hand it can provide damping (camber of wheels) and on the other it can lead to instability (flutter, Karmann vortices, shimmy of wheels).

In this respect it is worth mentioning that welded constructions have a very low internal damping, because there are no frictional forces involved as are in bolted and riveted constructions.

When incidental loads are to be accounted for, it will be necessary to evaluate the change in response due to the damage caused by these loads, in order to evaluate a possible reduction in endurance and/or the fail safe properties of the structure.

3.2. Distribution of forces

The distribution of the forces within the structure is the next step. This involves the static and the dynamic response of the elements of the structure. Linear elastic behaviour can normally be assumed to calculate the mechanical transfer functions, but the structure has to be scrutinized for important non-linearities, e.g. the lowering of resonance frequency due to a static compressive loading or the local (elastic) buckling of (thin) plate elements. Their influence under the assumed service- and environmental loads is to be determined.

In this respect the incidental loads are of paramount importance, because the damage involved is usually in the form of gross plastic deformation or maybe local fracture.

\(^1\) to be replaced by impulse, displacement, acceleration etc. when appropriate.
The result of plastic deformation is very often a change in eccentricity and in stiffness of some structural elements, so that both the static and the dynamic forces in the structure change. Note that every structural joint has redundancy.

Some remarks regarding the accuracy of the mechanical transfer functions are to be made:
- Structural damping is a quantity that has to be derived from past experience or from experiments. Especially the internal damping of materials and joints can show large variations.
- The stiffness of joints is a less known quantity and the behaviour in the elasto-plastic range is usually unknown.
- Structural members are not as straight and square as shown on the drawings, especially not when welding is involved. Secondary moments due to eccentricities are very difficult to access but will impair the endurance considerably.

3.3 Stresses and strains

From the forces that act on a structural element or joint the stresses are derived and used for the determination of the endurance either by calculation or by experiment.

A possible internal redistribution of static and dynamic stresses due to plastic deformations and dimensional changes after incidental loads is thoroughly to be considered.

Although it is a general experience that under laboratory conditions moderate positive overloads are beneficial for the endurance of the structure, this is by no means sure for high overloads, whether they are in tension or in compression. Compressive overloads cause usually an increased eccentricity due to the combined action of buckling and bending which results in increased bending stresses and a sharply decreased endurance.

4. Bases for service life estimates

4.1 Load spectra

Endurance calculations are based on load spectra that are derived from:
- measurements from similar structures
- estimations from the anticipated use of the structure and of the environmental loads
- a mixture of both.

The general procedure for generating a load spectrum is:
- measurements from a number of similar structures yield histograms of loadranges versus numbers of occurrence.
- from these histograms a probability density curve that also covers the low probabilities is developed, because measuring periods are usually short with regard to the life of the structure.
- determination of the omission level. For testing this level can be important, see par. 4.2, for a straightforward damage calculation it is not.
- determination of the truncation level, fig. 2 It is usual to clip at a load levels that occur 10 times per lifetime [1, 2, ]. For a fleet of
similar structures this means that this level is experienced several times by the majority of the fleet [3].

So the truncation level discriminates between the normal loads and the incidental loads.

The standard loading spectra for certain types of constructions that are available today are, except Gauss, for aircraft and aircraft engines, but several standard spectra for other applications are under development [1], viz.

- **WASH** - offshore structures
- **Walz** - steel mill drive
- **Wisper** - wind turbine
- **Carlos** - car components

The standard spectrum Gauss [4], fig. 3, is not attached to a certain type of construction. It is based on a Gaussian process and used for general fatigue investigations [1].

Furthermore many firms in the vehicle industry have developed their own (standardized) load-sequences, mainly for full scale testing.

From the loading spectrum the force- or stress spectrum for the structural element has to be deduced, see par. 3, so that the endurance can be calculated with the preferred cumulative damage rule provided that basic endurance data are available for material and shape of the structural element.

Verification tests on full scale structures in order to prove the validity of the calculations are usual in the aircraft- and in vehicle industry. Some insight into this type of testing as applied to vehicles can be gained from [5, 6].

### 4.2 Variable Amplitude Testing

A few remarks about Variable Amplitude testing should be made here:

- It is usual not to exceed the static design stress, $S_{max} = Re/SF$ where **SF** is the safety factor, usually $\approx 1.5$.

- When testing a 25 cps, an endurance of $2 \times 10^7$ is reached after 220 testing hours, which is certainly more than 10 days. However with the usual servo-hydraulic machinery 25 cps can only be obtained for the smaller type specimens. For large specimens 5 cps is a better average, and in the latter case $2 \times 10^7$ is reached after 2 months. For structures or a part of a structure the speed of testing is much lower than 5 cps.

- An advantage of V.A. testing is that the scatterband of the endurance is usually smaller than for C.A. testing because of the leveling of internal stress irregularities under the high stress ranges. It follows that the number of specimens per endurance curve can be reduced.
A reduction of the return period is sometimes possible by the omission of the lowest loadranges, either on the base of $\Delta K_{\text{threshold}}$ [7] or the constant amplitude fatigue strength $\Delta S_f$ (usual is $\Delta S_{\min} = 0.5 \Delta S_f$) or on the base of the damage they cause in relation to the total damage of the stress spectrum.

By limiting $\Delta S_{\min}/\Delta S_{\max} \geq 0.15$, a reduction in return period of a factor 5 to 10 can be obtained for a Laplace-type spectrum [1], but for a GAUSS-type spectrum omission levels of that magnitude do not really reduce the return period, see fig. 3.

From the above it is clear that there are severe limitations with regard to time and funds with regard to variable amplitude testing. It is therefore understandable that test results for endurances in excess of $2 \times 10^7$ are rather scarce.[8, 9]

For the presentation of results of V.A. tests the endurance is most usually plotted as the number of upgoing crossings of the mean-stress level. There are different opinions however about the significant stress parameter, and this may confuse a designer who is not acquainted with V.A. testing.

A recent discussion about this subject is given in [10].

However it should be remembered that random loads can never be represented by one parameter unless a completely defined standard spectrum is used.

Further it should be remembered that for statistical defined distributions there is a fixed relationship between the different statistical parameters and conversion, e.g. for the application of Miners Rule, is not too difficult.

4.3 Basic data

Basic data for cyclic properties, endurances, crackgrowth and fracture toughness are available for a large number of metals and alloys [11-14] and in BS5400 for welded joints. Unified endurance data [15] for mechanically notched specimens from a variety of steels are also available [16].

Design endurance curves for welded joints as given in many national design codes, except those given in BS5400, cannot be regarded as basic data because the factors of safety etc. incorporated into these curves are not too well defined.
5. Designing against random loading

5.1 Machinery versus structures

In designing against fatigue there is an essential difference between designing machinery and designing structures. For machinery the number of service load cycles involved during their life is of the order of $10^9$ or more, and even loads that occur with a probability of $10^{-4}$ or $10^{-5}$ are to be below the fatigue limit if the machine is to survive, because the service loads are severely truncated either by the users-manual or due to the available power. So the incidental loads are dominated by misuse and lack of maintenance.

It follows that machinery is designed against the fatigue strength and that surface conditions, surface treatments and (artificial) residual stresses are of paramount importance. Furthermore replacements of underdesigned components by upgraded ones is usually possible. Fail-safe aspects are - if applied - rather simple.

For constructions the picture is entirely different because environmental and/or service loads vary strongly in magnitude and designing on the base of the fatigue strength is very often not feasible. Although it is virtually impossible to design against uncontrollable loads as are the environmental ones, it is possible to design against the damage caused and minimize the risks involved.

5.2 Design targets for constructions

The principle target in designing a construction against random loading is to ensure the integrity of that structure up to and beyond the anticipated service life.
- A note about "beyond": In the world there are numerous railway bridges of all sorts that are more than 100 years of age. The number of large commercial aircraft that are approaching or have exceeded their design life is steadily increasing.

In this respect are targets of paramount importance:
- fail-safe quality
- overload capacity
- safe endurance
These 3 aspects are of course heavily interrelated.

5.2.1 Fail safe quality

Fail safe in the narrow sense means that failure (by fatigue) of one element of a structure does not lead to the destruction of that structure and that such a failure is recognized in time. This means that there is to be more than one path to transfer the forces through the structure, and in general this is obtained by making the structure redundant. Whether full redundancy can be reached depends on the type of construction. Parts that are not are to have a higher reliability.

Fail safe in a broader sense means that (very) high incidental loads or the premature failure of a structural element may render the structure unfit for purpose, but will not lead to complete destruction or disaster.
The platform Alexander Kieland is an example of a structure that was not fail-safe. Failure of one brace led to complete destruction and disaster.

Weak spots are to be brought forward by
- Failure mode-
- Limit load-
- Plastic shake down-analysis
and it has to be decided whether these are to be
- downgraded,
- maintained or
- upgraded,
in order to improve the fail-safe characteristics of the structure.
As fatigue cracks may have developed during previous use, these have to be taken into account in the analysis.
It is clear that the fracture toughness, not only of the material as such, but of the cracked structural element can influence the failure mode considerably.

5.2.2 Overload capacity

From the above it follows that it has to be decided what magnitudes or severities of incidental loads the structure should be able to stand without significant damage with regard to its future life. It follows that - eventually after repair - incidental loads up to that level must not reduce the fatigue life.
The lower boundary of the incidental loads are layed down by the maxima (truncation level, see par. 4.1) of the load spectrum to be used for the endurance calculations and/or tests.

The argument that the higher peakloads cause the greater part of the retardations and lead to an over-estimation of the endurance for those items of the fleet that may not encounter these high loads, is correct only to a certain level (of the peakloads). This level is essentially determined by the condition that the stress- and strain distributions before and after these peak-loads are comparable and no significant dimensional changes are induced.

Now the experience that when testing under random loading:
- the scatterbands are smaller
- high positive load excursions increase the endurance considerably [8]
- negative loads do not have much effect on the endurance,
are all due to plasticity effects that lead on the one hand to relaxation of varying distributions of internal stresses in a complex joint and on the other hand to crack closure phenomena [17].

Higher positive or negative peak loads can cause excessive plasticity in severely notched structural elements or joints and lead to appreciable changes in the stress distributions (e.g. bolted joints, see par. 6.3) or in the always existing excentricities. The very high deviations of experimental results from predictions calculated according to Miners Rule as recorded in the past can often be traced back to these effects.
The argument that the safety factor on the maximum load covers the influence of incidental higher loads has only very restricted value, because this factor has to cover:

- uncertainties in the load-to-stress calculations
- variations in material properties
  - material dimension
  - production quality

In this respect it should also be noticed that the just mentioned types of variations are not really at random because systematic influences enter due to the production process of material and parts, e.g.:

- heat treatments for smaller parts, e.g. bolts, are carried out en masse.
- n.c. machinery produces parts of almost identical shape.
- adopted welding procedures can yield systematic low or high fatigue properties.

5.2.3 Estimation of the Service life

For the estimation of the service life - that is the calculation of the endurance -, generally use is made of Miners Rule. Miners Rule [18] was stated at a time (1945) that only limited results from program tests were available and results from variable amplitude tests were not available. Today, more than 40 years later, this has changed drastically, and instead of using $\sum N_i/N_1 = D = 100\%$, as a first approximation for the number of cycles to failure, experimentally established values of $D$ from tests under standardized stress spectra can be used.

The value of $D$ depends as well on the type of stress spectrum as on the type and size of the structural element or joint. This latter method is known as the Relative Miner Rule and yields certainly much more realistic results than Miners Rule, if only because sequence effects are better taken into account.

However, to the knowledge of this author, no compilations of experimentally determined values of $D$ are available.

To obtain a safe endurance, the influence of incidental loads has to be assessed either from:

- previous experience in the field
- tests on the structure or on larger parts of it
- very carefully designed tests on smaller parts (boundary conditions).

In this type of testing artificial damage can be introduced.

This author is aware that about this subject there is not much information available in the open literature.

6. Design considerations

6.1 High strength material

For structural steels the fatigue crack propagation rate and the threshold value of the stress intensity for macrocracks are apparently not influenced by the tensile strength of the material.

For design purposes are used:

$$da/dn = 0.2 \times 10^{-12} \Delta K^2$$

80 < $\Delta K_{th}$ < 100

$[N, mm]$ (no crack closure)
It follows that when the endurance is dominated by macrocracks the use of steels with a higher strength does not improve the life under constant amplitude conditions. This may be true, but the advantage in using H.S. steels with regard to incidental loads is clear. The (gross) plasticity induced by overloads is reduced considerably and this results in a greater reliability of the endurance calculations, provided of course that the general stress level is derived from fatigue considerations and not from the maximum design load.

6.2 Quality of welded constructions

Design codes for welded connections do not distinguish between welding processes or consumables, and in general the same holds for the specifications on drawings for welded constructions. Weld inspection procedures do care for undercuts and lack of fusion at the weld toe and for the size of the overfill (still called reinforcement!), but do not care much for e.g. the shape of the weld toe. However, it should be admitted here that even for a fatigue expert it is hardly possible to judge the quality of a weld with regard to fatigue.

Consumables are qualified on the base of their own merits and for the base material only the hardness and the transition temperature of the heat affected zone seem to be important. It follows that every construction firm or even welder can choose his own favorite welding process and consumable, and that the fatigue properties of a welded joint are at random [19] and unsuitable for a reliable fail-safe design because it cannot be estimated which joint will fail first from fatigue.

That systematic research can reveal systematic trends is shown in [20]. From this investigation it appeared that for a high grade TM steel (240 HV5) the hardness of the weld was very important. A hardness of the weld of about 40 HV5 above that of the plate material yielded a fatigue strength $S_f = 150 \text{ N/mm}^2$ at $R = -1$ for an axial loaded transverse butt weld in 6 mm plate material, fig. 4. The results of random load tests on these joints are given in fig. 5.

Some recent results for transverse butt welds in 9 mm T.M. steel of similar grade are shown in fig. 6. In this figure the lower S-N curve holds for S.A. welding without edge preparation and the upper one for Pulsed Mig welding with an edge preparation of 2x30° bevel. The appearance of the weld profiles was the same, the hardnesses of the consumables were comparable as were the heat-inputs. However, as no edge preparation was applied for the S.A. weld, the hardness of the weld went down to 220 HV5 whereas the hardness of the Pulsed Mig weld was 260 HV5. (It should be noticed that the HAZ of this TM-steel shows softening to 190 HV5.)

As the difference in fatigue strength is a factor 2.4, the difference in endurance under random loading could be a factor 10 or more.

Weld improvement techniques are becoming of use now. A review with regard to off-shore applications is given in [21] and improvement techniques will also be the subject of a lecture to be delivered during this Symposium. Again with regard to reliability an example is discussed below.
Fig. 7 shows the endurance for C.A. loading for a longitudinal non-loadcarrying fillet weld in a high grade TM steel. The drawn lines are for end welds (in this case also the tack welds) made manually. The dashed lines are for end welds made with the Pulsed Mig process. Both types of end welds were also tested in the TIG-dressed condition (thick lines) and it appears that the influence of TIG-dressing depends strongly on the quality of the weld. Note that the improvements and the differences remain under R.A. loading, fig. 8.

Fig. 7 also shows that after improving the weld toe, another failure mode, viz. cracks from the root of the end weld, appear in the C.A. tests. This failure mode did not show up in the R.A. tests because these tests did not go beyond \( N = 2 \times 10^7 \). However it can be expected that this failure mode will appear at endurances of \( 10^8 \) or more.

From the examples given above, one may question the possibilities to design fail safe qualities or a safe endurance into a structure when the basic fatigue resistances show large variations.

It follows that there is a strong need of quality control based on systematic screening tests of welding parameters with regard to the fatigue properties of welded joints in different grades of steel.

### 6.3 Friction grip joints

Friction grip joints transfer the loads by the frictional forces between the mating surfaces. The high clamping forces necessary are delivered by high strength pre-tensioned bolts. The coefficient of friction is increased to more than 0.5 by the application of a high friction, corrosion resistant, coating on the mating surfaces. However, when in the V.A. load spectrum there are peak loads that exceed the frictional forces by say no more than 10%, the coefficient of friction is reduced to about 60% of its initial value because of repeated slip. This holds when high friction primer is used [20] but probably also for other high friction coatings. Fig. 9 shows a registration of force versus displacement during a quasi-static repeated slip test of a friction grip joint.

So overloads on these joints lead to a redistribution of forces in the joint and also to hammering and it is clear that any endurance calculation based on friction-grip conditions fails.

### 6.4 Tubular joints

Within the European programs on steel in marine structures, subsidized by the ECSC, much research has been carried through about the endurance of tubular joints as used in off-shore platforms [22, 23]. The value of these investigations for the integrity of offshore structures can hardly be overestimated. However, most tests on tubular joints were carried out with the cord loaded only by the forces from the braces, and most tests were ended when or shortly after the cracks had grown through the wall.

With regard to the endurance this approach is satisfactory because, after the crack has penetrated through the wall, the remaining life is small.
However, insufficient insight is gained about the crack growth in the chord, being the major load carrying member. Fig. 10 shows 3 examples of crack paths obtained from a rather stiff triangulated lattice girder (chord & 168x7 mm, brace & 76x5 mm, brace length 1100 mm) that was tested under C.A. loading. With regard to fail-safe characteristics these photographs speak for themselves.

In this respect the increasing use of cast nodes (e.g. the jacket of the Veslefrikk platform) is very advantageous. Due to their favourable shape and the inherent high fatigue resistance, the critical sections are situated at the outside of the node, so that fatigue failure of e.g. a brace does not lead to a weakening of the node itself. A further advantage of cast nodes is that overloads do not change the shape, while overloads on a welded node will cause plastic deformations of the wall of the tubes, fig. 11. The resulting out of roundness then causes a redistribution of the stresses and a change in secondary bending stress ranges that may impair the endurance.

7. Concluding remarks

From the foregoing paragraphs it follows that loads of very low probability - here called incidental loads - may have a detrimental effect on the endurance of a structure. Therefore the design has to be based primarily upon these loads and their effect on the behaviour of the structure afterwards has to be estimated. - For the extreme loads this regards the fail-safe qualities and includes the selection of failure modes. - For the (very) high loads this regards the redistribution of static stresses, but also and of paramount importance, the changes in dynamic stresses due to geometrical changes from plastic deformations. So the design should be such that these changes remain as small as possible and that the generation of non linearities is avoided. It was discussed in par. 6.4 that for tubular joints cast nodes are far superior to welded nodes.

However structural reliability, based on a selection of the most critical sections is impossible without knowledge of the endurance of these critical sections. Welded joints are very unsatisfactory in this respect because essentially only the lower bound of the fatigue resistance (so macro crack propagation) is known. In par. 6.2 it is shown that welding parameters are of real importance and an investigation (by a round robin) into the systematic influence of welding processes and consumables on the fatigue resistance could lead to greatly improved possibilities for a reliable failure analysis and safe endurance of welded constructions.
References.


[22] ------. Int. Conf. on Steel in Marine Structures, Paris 1981. Report EVR 7347 DE, EN, FR.

SERVICE

ENVIRONMENTAL

INCIDENTAL

$P \geq 1$

$P \geq 1$

$0 < P < 1$

TAILS OF THE DISTRIBUTION

INCIDENTS (ACCIDENTS)

LOCAL FAILURE

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Fig. 1 Types of loading

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Fig. 2 Truncation and omission levels for load spectra
Fig. 3  Spectrum and 3 sections of the stress-time history with different irregularity factor for Gauss [1] (courtesy ASTM)
Fig. 4  Endurances for a transverse butt weld, FeE560TM, 6 mm plate.
Axial loading, C.A. [20]

Fig. 5  As fig. 4, [20].
V.A. loading, Gauss I=0.99 and 0.7,
(S max = 5.26 S rms)
Fig. 6 Endurances for a transverse butt weld, FeE560TM, 9 mm plate.
C.A. loading.

- S.A. welding
- Pulsed Mig welding
- BS5400 Class D
Fig. 7  Endurances for a longitudinal n.l.c. fillet weld. Fe560TM, 6 mm plate.
Axial loading, C.A.

Fig. 8  As fig. 7
V.A. loading, Gauss, I=0.99
Fig. 9  Force versus displacement for a friction grip joint, high friction primer (courtesy DAF Trucks).

Fig. 11  Local deformation after overload (schematic).
Fig. 10 Different crackpaths for tubular joints.
IMPROVEMENT TECHNIQUES

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Summary

This paper reviews some of the commonly used weld improvement techniques and their effect on the fatigue performance of welded joints. The influence of various factors such as base material strength, variable amplitude service loading and specimen size are examined. The relationship between size effects and local stress concentration is given special consideration. Problems associated with the interpretation and consolidation of mainly small scale test data and the transformation of such data to design information for large scale structures are discussed.

1. INTRODUCTION

Two decades of research on improvement methods has produced a large body of test results and considerable insight into the factors that influence the fatigue life of test specimens with improved welds. While some progress has been made in digesting this information and translating it into practical design data, relatively little progress has been made in applying improvement techniques to welded structures on an industrial basis, the major obstacle being that current codes of practice provide only limited guidance for the improvement of fatigue strength.

Modern steelmaking techniques have led to the production of structural steels with excellent weldability and fracture characteristics. As a result brittle fracture has been eliminated as a service problem, also for the higher strength steels with yield strengths up to about 500 MPa that are now increasingly being used in offshore structures. A particular problem related to the use of higher strength steels is that the fatigue strength does not increase with static strength for as-welded connections. A general problem is that increasing size leads to lower fatigue strength and the reduction is apparently related to the geometry of the joint. Thus size effects should influenced by improvement methods that reduce stress concentrations. However, there is still many uncertainties regarding the relationship between size effects and joint geometry.

Both the problem regarding improvement methods and higher strength steels and size effects are addressed in this paper. An attempt has been made to assess the state of the art and to present some possible solutions to the problem of deriving design S-N curves for improvement methods from the experimental data available.
2. IMPROVEMENT METHODS - OPERATING PRINCIPLES

The low fatigue strength of welded connections is generally attributed to the very short crack initiation period which is generally found to be in the range of about 10 to 30 percent of the total life, depending on the method of observation and definition of the initial crack. Comparing this with a crack initiation period of more than 90 percent typically observed for smooth specimens tested at low stresses there is obviously scope for a substantial life increase by delaying crack initiation. The main factors contributing to the low crack initiation life are shown in Fig. 1. This figure also gives an indication of how to extend crack initiation life, namely by:

a) reducing the stress concentration factor of the weld
b) removing the crack-like defects at the weld toe,
c) removing the harmful tensile welding residual stresses or introducing compressive stresses

Depending on the main principle on which the increased fatigue strength is based weld improvement methods can be placed in two broad groups:

1. Weld geometry modification methods
2. Residual stress methods

Various methods that have been investigated in the literature are listed in Fig. 2. In addition two other methods have been tried in the last few years; these are water gouging and laser remelting. However, the two methods are still very much experimental in nature and have to the author’s knowledge not been employed in industry. The following methods have reached a more mature stage in the sense that they are used in industrial applications:

1. Weld profile control, i.e. performing the welding such that the overall weld shape gives a low stress concentration and the weld metal blends smoothly with the plate.
2. Special electrodes with good wetting characteristics to give a favorable weld toe geometry
3. Weld toe grinding, using either a disk grinder or a rotary burr tool
4. Tungsten inert gas (TIG) remelting of the weld toe region
5. Hammer peening of the weld toe region
6. Shot peening

Grinding and TIG remelting methods may be classed weld toe geometry modification methods, whereas hammer peening and shot peening are residual stress techniques. Particularly large improvements may be obtained when techniques from the two groups are combined.
3 SOME IMPROVEMENT METHODS AND THEIR EFFECT ON FATIGUE STRENGTH

3.1 Improved welding techniques

Weld profiling and the use of special electrodes are methods that are integral part of the welding process itself. This is obviously attractive from a production point of view since there is no need to come back with a different type of equipment for a final treatment of the weld, which would increase costs and worsen quality control problems.

The AWS improved profile.

In the American Welding Society Structural Welding Code [3] a low stress concentration factor is sought by controlling the overall shape of the weld to obtain a concave profile and requiring a gradual transition at the weld toe. The "disc test" or "dime test" shown in Figure 3 as specified by AWS is used to ensure an acceptable weld. If the weld does not pass the disc test remedial grinding at the weld toe or at the interbead notches has to be carried out. If profile control is carried out the designer can use the X1 curve in Figure 4, if not he has to use the lower X2 curve.

The baseline data for the AWS improved profile X1 curve were obtained from tests on tubular joints [4] in the early seventies; apparently no tests were made on planar joints. Recently some tests have been made in France [5] and Norway [6,7] on transverse fillet welds with controlled profile. The test results in Figures 5-7 show very similar improvements in fatigue strength, generally between 25 and 30 percent at long lives. Tests on tubular joints with improved welds have given mixed results, and in the early UKOSRP and ECSC programs no effect of weld profile control was found, see Figure 8, whereas later tests by Dijkstra and Noordhoek [8] gave three times longer fatigue lives for the improved joints. Also some recent French tests on tubular joints [9] have indicated the beneficial effects of weld profile control. However, the interpretation and comparison of test results is hampered by the inadequacy of the ECSC hot spot strain concept to allow for variations in weld leg length in a meaningful way.

Special electrodes

Specially developed electrodes with coatings that have good wetting and flow characteristics have been used in Japanese test programs aimed at improving the fatigue performance of high strength steels of 500 to 800 MPa yield strength [10-14]. These electrodes are also understood to have been widely used in the construction of high strength steel bridges. The electrodes give a smooth transition at the weld toe and a reduction of the calculated stress concentration factor, typically from around 3 to 1.2-1.5 for fillet welds [14].

The improvements reported in the Japanese tests were from 50 to 85 %, the largest increases in fatigue life being reported for the highest strength steels. Some tests on T-joints made recently in Norway gave improvements of approximately 25 % [7]. The main doubts about the special electrodes concern their use in positional welding where the easy flow of the filler material may be a disadvantage. A related technique is to use special electrodes only for the finishing passes at the weld toes. Improvements of 60 and 80 % have been reported [15,16].
3.2 Grinding

Grinding (Fig. 9), can be carried out with a rotary burr grinder or disc grinder, the former requiring much more time and therefore incurring higher costs. To ensure the removal of slag intrusions grinding has to be extended to a depth of minimum 0.5 mm below the bottom of any visible undercut [17]. The lower stress concentration factor and the removal of crack-like defects at the weld toe generally give large increases in fatigue life, typically from 25 to 100 percent at long lives ($N > 1$ million cycles). However, the scatter is large, particularly for disc grinding which may be difficult to perform in confined areas; also an inexperienced operator may inadvertently remove too much material. In an extensive survey of published results for the two grinding techniques, Booth [18] found improvements from 10 to 160 percent at 1 million cycles, the largest improvements were obtained for QT steels of approximately 700 MPa yield strength. In an earlier statistical analysis of test results, Olivier and Ritter [19] found variations ranging from no improvement in fatigue strength to more than 250%.

Grinding is currently the only improvement method allowed in European codes for offshore structures [17,20], however, the higher fatigue strength is not intended for use in initial design, instead grinding may be used as a remedial measure if the design life is shown to be inadequate at a late stage during design or construction [17,20].

3.3 Weld toe remelting

Remelting of the weld toe using either TIG or plasma welding equipment generally results in large gains in fatigue strength, for several reasons. Firstly, the smoother weld toe transition reduces the stress concentration factor; secondly, slag inclusions and undercuts are removed, and thirdly, in some Japanese publications the higher hardness in the heat affected zone is claimed to contribute to the higher fatigue strength.

Plasma dressing generally tends to give better results than TIG dressing, this is ascribed to the higher heat input and the larger pool of melted metal obtained with plasma dressing.

TIG dressing

Standard TIG dressing equipment is used, usually without any filler material. For the older type C-Mn steels (e.g. St 52) with a relatively high carbon content a second TIG run was necessary to temper the first run at the toe [21], see Figure 9. The second run also contributes to a better weld toe geometry. This hardness problem associated with TIG dressing of C-Mn steels is eliminated with the use of modern low carbon steels. TIG dressing is somewhat sensitive to operator skill, the weld and plate must be clean to avoid pores. Millington [22] has defined optimum conditions for TIG dressing.

The magnitude of the improvement depends as for most improvement techniques primarily on the joint severity and base material strength. Improvements range from about 10% for butt welds in mild steel plates to about 100% for fillet welded high strength steels have been reported in the literature [15,16,21,23-27]. Some test results in terms of fatigue strength at 2 million cycles are shown in Figure 10.
Plasma dressing

Plasma dressing is similar to TIG dressing, the main difference being the higher heat input, about twice that used in TIG dressing, and a wider weld pool. The latter tends to make plasma dressing less sensitive to electrode position relative to the weld toe, and the resulting improvements in fatigue strength are generally larger than for TIG dressing at about 60 to 200%, particularly for higher strength steels [28,29], but smaller improvements of about 40% have also been reported [23].

3.5 Residual stress methods

Some improvement in fatigue behavior is obtained by removing welding residual stresses by postweld heat treatment, especially if the applied load cycle is wholly or partly in compression. However, the largest benefits are obtained if compressive residual stresses are introduced. The more commonly used residual stress methods are hammer peening and shot peening.

Hammer peening

Hammer peening is carried out with a solid tool with a rounded tip of 6-14 mm radius. A similar technique consists of using a wire bundle instead of a solid tool. Both types of tool are normally pneumatically operated. The solid tool gives a far more severe deformation and gives better improvements than either wire bundle or shot peening [30].

Optimum results for hammer peening are obtained after four passes, giving a severely deformed weld toe, with an indentation depth of about 0.6 mm, providing a simple inspection criterion [30].

Like burr toe grinding, hammer peening is a noisy and tedious operation and has perhaps for this reason not attained widespread use. The improvements are among the highest reported, see Figure 10. Most test results show larger improvements for higher strength steels [30,32], but some recent tests performed at the Welding Institute on transverse fillet welds indicate no correlation between base material strength and the degree of improvement [33].

Shot peening

In the shot peening process the surface is blasted with small steel or cast iron shots in a high velocity air stream, producing residual surface stresses of about 70 to 80% of the yield stress. Assessing the quality of the treatment entails time consuming residual stress measurements, instead the intensity or the degree of surface plastic deformation is determined by Almen strips, which are small steel strips attached to the surface of the component. The curvature developed in the strip is a measure of the peening intensity. A second parameter is area coverage. 100% coverage is obtained when visual examination at 10X magnification of the surface shows that all dimples just overlap. The time required to obtain 100% is doubled to obtain 200% which normally is specified. A major advantage of shot peening is that it covers large areas at low costs.

Results from fatigue tests on shot peened welded joints show substantial improvements for all types of joints, the magnitude of the improvements varying with type of joint and static strength of the steel. Typical
results are 30 to 100% increase in fatigue lives in the long life region; however, at shorter lives ($N < 10^6$ cycles) the improvements tend to disappear. Tests in sea water show that the improvements are retained even under freely corroding conditions [5, 34].

High peak loads in variable amplitude loads sequences may be assumed to relax the residual stresses and reduce the efficacy of such methods, but German results have shown no such adverse effects [34].

### 3.6 Compounding

The combination of two improvement methods, particularly a weld geometry method and a residual stress method, are likely to give large improvements. One example is full profile grinding and hammer peening which resulted in the fatigue strength of fillet welds in mild steel being restored to that of the base material [35]. More common combinations are grinding and shot peening [6, 7], and AWS weld profile control and shot peening [5-7]. In such cases the resulting improvement may be double that of a single method.

### 4. APPLYING IMPROVEMENT METHODS TO REAL STRUCTURES

Most current knowledge on improvement methods have been gained from tests on small scale planar specimens. When considering the application of weld improvement methods to actual structures the differences in fatigue behavior has to be evaluated. One important factor is size. In a large structure long range residual stresses due to forcing the members together are present and influence fatigue life. Another consideration is the existence of alternative failure sites. Obviously no improvement can be expected for a joint with load-carrying fillet welds whose toe regions are ground or TIG dressed if the untreated joint is as likely to fail from the root as from the toe; the failure would only be shifted to the root. Similar problems were encountered by Minner and Seeger [24] who tested beams that were TIG dressed, in their tests only modest improvements were obtained due to premature failures from hydrogen cracks. In tests by Bergquist and Sperle [36] on coverplated beams the failure location was moved from the toe to the root of the coverplate fillet welds after TIG dressing, but the improvement was still substantial at 40% in fatigue strength.

In contrast to small joints where the peak stress is limited to the weld toe, the peak stress region in a large multi-pass joint may include several weld beads and cracks may initiate anywhere in this highly stressed area. This effect is even more pronounced for welds in tubular joints with high beta ratio (brace diameter to chord diameter ratio), as illustrated in Figure 11. A stress distribution as in Figure 11 means that not only the weld toe but the entire weld needs to be treated.

A problem related to tubular joints with low beta ratios is the very steep stress gradient at the weld toe which is caused partly by the global geometry. If the weld leg length is reduced, e.g. by grinding as indicated in Fig. 12 the resulting peak stress may well be higher and the resulting improvement could be marginal or non-existing.

Improvement methods that reduce the stress concentration factor are likely to have a beneficial influence on the adverse effect of plate thickness [6]. This aspect is discussed further in the next section.
5. IMPROVEMENT METHODS AND DESIGN RULES

5.1 Current design rules incorporating improvement techniques

As noted in section 3.1 the weld profile improvement method is included in the AWS/API design rules in terms of the X1 curve that may be used generally if profile control is carried out, otherwise the lower X2 must be used. The two curves intersect at a life that is somewhat less than $10^4$ cycles, i.e. the improvement is lost at this life.

In the UK Department of Energy rules S-N the curves for all types of joints can be moved by a factor of 1.3 on strength (2.2 on life) if grinding is carried out [17]. Thus the two curves are parallel, and the improvement applies also in the low life/high stress region, contradicting most test data which tend to show very small or no improvements at all in this region, giving intersecting as-welded and improved S-N curves, as exemplified by Figure 10.

The Swedish design code for welded structures consists of 10 S-N curves each of which is identified by its $K_x$ factor, see Figure 13. The code also includes a weld quality system containing four basic classes plus an additional class designated U for improved fatigue strength. Use of the improved class requires that:

- undercuts, weld reinforcements, penetration beads, un-filled grooves and root concavities must blend smoothly with the base material
- incomplete root penetration is not permitted
- arc strikes must be avoided or removed

The use of improvement techniques such as grinding, TIG dressing and hammer peening is permitted to obtain the highest quality class. The combination of weld geometry, probability of survival level, and weld quality thus determines the S-N curve to be used. The Swedish system of S-N curves is similar to the British rules insofar that employing an improvement method leads to a parallel shift of the S-N curve.

5.2 Improved welds and size effects

Size effects in notched components are generally attributed to three origins [38], i.e. a technological size effect, a statistical size effect or a geometrical or stress gradient size effect.

Technological size effects result from differences in production parameters, generally leading to lower mechanical strength for the thicker parts. Also residual stresses and surface quality may vary with thickness.

Statistical size effects arise from the higher probability of encountering a large defect in a large volume of material compared with a smaller volume.

Geometric size effects arise from the stress gradient at the notch root. Even if geometric scaling is maintained the stress gradient is steeper for the thicker part and a crack will grow in a higher stress field. If geometric scaling is not maintained which is usually the case for welded
joints, the stress magnification factor increases with thickness [4,38-39], see Figure 14. Morgan [40] demonstrated this effect by strain gage measurements in the toe region of welds.

Fracture mechanics calculations [4,39,41] have shown that the influence of thickness increases with the SCF of the joint. A statistical analysis of published data on size effects in welded joints gave a size exponent of $n = 0.33$ for as-welded joints and $n = 0.20$ for improved joints, where $n$ is the size exponent $n$ in the thickness correction equation

$$S/S_0 = \left(\frac{t_0}{t}\right)^n$$

(1)

This tendency to get a smaller influence of size for unnotched or mildly notched parts has been shown to exist for mechanical components [38], and the following relation between $n$ and the SCF has been proposed [38]:

$$n = 0.1 + 0.14\log K_t$$

(2)

where $K_t$ is the stress concentration factor. Eq. 2 is illustrated in Figure 15 together with some values of $n$ calculated from test data. Eq. 2 reduces to the SAE rule ($n = 0.1$) in the case of unnotched parts, whereas a stronger size penalty is imposed on notched parts.

5.3 Future modifications to design rules

The current status of improvement methods is not satisfactory as several methods with proven ability to improve the fatigue strength of a large variety of small scale specimens as well as large structural components are not included in design rules. Moreover, the European rules [17,20-37], which give the same improvement at all lives, are not consistent with test data which indicate that the largest improvements are obtained in the high-cycle region, and very small or no improvements in the low-cycle region ($N < 10^4$ cycles). Thus, a more logical approach would be to increase the fatigue strength near the fatigue limit, say at $N = 2 \times 10^6$, by 30% (for ground joints) and assign an inverse slope of 3.5 to the curve at shorter lives. At longer lives an inverse slope of 5.5 would be used to allow for cumulative damage, see Figure 16. This curve for improved joints intersects the as-welded curve at a life of about $10^4$ cycles. This amendment would also bring the S-N curves more in line with the AWS/API curves shown in Figure 4.

Secondly, both theoretical and experimental results indicate that size effects are less severe for mildly notched parts than for the more severe joints with very short crack initiation lives. Thus a size exponent of 0.2 would probably be adequate for low SCF joints like simple butt welds or T-joints with small attachment thicknesses. For the higher SCF joints e.g. Class F and lower, a size exponent of $n = 0.33$ would be more suitable [38]. For improved welds an exponent $n = 0.2$ for all weld classes would probably be adequate.

Life predictions that include a crack initiation stage using local stress strain concepts plus fracture mechanics methods for the crack growth stage have given reasonably accurate life estimates for improved welds [6], and support the experimental observation that the fatigue lives of improved welds generally increase with base material strength. Thus a third, and perhaps more controversial modification to the design
rules would be to allow higher fatigue strength for higher strength steels, but more data has to be collected before specific recommendations regarding the degree of improvement can be made.

An effort is now being made within the International Institute of Welding's Commission on Fatigue behaviour of welded components and structures to collect data on improvement methods, with the aim of developing recommended shop practices and design guidance for improvement methods.

6. CONCLUSIONS

Improvement methods have been examined in many investigations over the last 15-20 years and the following main conclusions may be drawn from a survey of this work:

1. Experimental evidence and fatigue life predictions indicate that substantial increases in fatigue strength can be obtained consistently when improvement methods are used. However, the full potential of weld improvement method can only be obtained if premature failures from other locations e.g. the weld root, can be avoided.

2. The degree of improvement is generally larger for higher strength steels than for mild steels.

3. There are strong indications from experiments and theoretical analyses that size effects are lower for low severity joints, implying that size effects are mitigated by weld improvement methods that reduce the local stress concentration.

4. The problems of quality control are similar to those involved in the welding process itself. The question of employing an improvement method is therefore related to costs and the benefit allowed in design rules.

7. RECOMMENDATIONS FOR FUTURE WORK

There is still a need to generate more data on improvement methods, particularly regarding the application of these methods to large scale structures. But a large body of data is available and an international effort is necessary to convert the research information into design codes and design guidance. It is therefore recommended that all member countries contribute to the IIW work on collecting and interpreting data on improvement methods.

REFERENCES


[37] Swedish regulations for welded steel structures 74 StBk-N2, National Swedish Committee on Regulations for Steel Structures, 1974.


Figure 1. Factors affecting the fatigue strength of welded joints [1].

Figure 2. Classification of some weld improvement methods [1].
No special finishing

Dime test to be applied to weld toes (A) and weld face irregularities (interpass notches)

Weld toe angle 135° min
R=1/2 except that 84H-25 mm
135° min

Coin or disc with radius R

1 mm wire shall not pass

Figure 3. The AWS improved profile and "dime test".

Figure 4. The AWS/API design curves with test data [4]
Figure 5. Improved profile test results for planar joints, Bignonnet et al. [5]

Figure 6. Improved profile test results for planar joints made of low carbon micro-alloyed steel of 370 YS, Haagensen et al. [6]

Figure 7. Improved profile test results for planar joints made of QT steel of 520-540 YS; Slind [7].
Figure 8 Improved profile test results on tubular joints

Figure 9 TIG dressing, a) single run, b) with additional temper run
Figure 10 Comparison of results obtained by some improvement methods, data from Refs. 31-32.

Figure 11 Stress distribution cross weld in a tubular joint, schematic

Figure 12 Stress distributions in tubular joint in a tubular joint before and after grinding, schematic
Figure 13  S-N curves in the Swedish design code [37]

Figure 14  Increase in calculated stress concentration factor with plate thickness due to lack of scaling of the weld toe geometry
Figure 15 Variation of size correction exponent $n$ with SCF for geometrically similar machined specimens [38]

Figure 16 Possible construction of design S-N curve for an improved joint
EVALUATION OF EXISTING STRUCTURES

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Summary

In this paper especially the evaluation of railway bridges concerning fatigue is considered.
The design loading scheme of the UIC is used as a reference to compare the load carrying capacity of existing bridges designed according to different loading schemes and different allowable stresses. Stress spectra of a standard train traffic with stress ranges in proportion to the stress range due to the UIC-loading are the base of a first step of the fatigue life estimation.
The influence of identification of SN-curves and the composition of the train traffic is shown.
As a further approximation a fatigue assessment procedure is shown using data of annual tonnage.
To verify the magnitude of calculated stresses in-situ measurements are mentioned.
Inspection procedures have to be established if the remaining life is negative or uncertain.
Some examples are given of the calculation of inspection intervals using fracture mechanics.
The risk of brittle fracture of some kinds of old steel used in riveted and welded structures is pointed out.
The fatigue behaviour of some special structural details is shown.

1. Introduction

From a point of view of evaluation existing structures concerning fatigue, especially railway bridges and highway bridges have to be considered.
Still there are steel bridges in service constructed in the beginning of the 20th century and also some at the end of the past century.
In general the bridges of that time are riveted structures and have been designed according to lower live loads than at present.
Also the properties of the applied steel differ from modern steel.
In the beginning the allowable stresses were dependent on the span of a bridge without taking into account any impact factor.
Later on impact factors were introduced in connection with fixed allowable stresses.
After that the allowable stresses became dependent on the ratio of the maximum and minimum stress in a bridge member.
The development of knowledge about fatigue of welded structures took a lot of years, and in the meantime welded bridges were designed according to different specifications earmarked by the state of the art at the time of construction.
When time went on the live load of bridges increased and the question was posed about safety and the expected remaining life of the existing bridges.
In general for bridges a design life of hundred years is adopted. While construction of steel bridges began over one hundred years ago, the question of remaining life becomes apparent for a number of bridges.

2. Fatigue strength of riveted structures

In the past time the fatigue strength of riveted structures was evaluated from test results determining the value of the stress at 2 million cycles, assuming that this stress is a limit below which fracture does not occur. While scatter was not taken into account the evaluated stress was more or less a mean value.

To achieve a certain probability of failure in fatigue calculations it is necessary to consider the phenomenon of scatter in fatigue test results. That is why recently all available data concerning fatigue tests on riveted connections have been re-analysed. It was found that in general the ECCS-curve 71 is a conservative lower bound of the fatigue strength.

The fatigue strength of riveted structures is influenced by several factors:
- clamping force
- bearing stress
- shear stress in the rivets
- stress ratio $R = \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}}$
- excentricity in the connection.

Reduced clamping force and high bearing stresses do decrease the fatigue strength very strongly. High shear stresses in rivets can cause fracture of the rivets and also reduce the fatigue strength. However when connections have a good clamping, low bearing stress and no excentricities, about 90% of the results are covered by the ECCS-curve 90 [3].

Figure 1 gives an idea of the effect of the mentioned influences. There is not a great difference between the fatigue strength of riveted connections made of steel and those made of wrought iron.

3. Fatigue strength of welded structures

Recently many research has been carried out about fatigue of welded structures. It was recognized that the stress range is the governing parameter for fatigue. Originally this was also stated by Wöhler in 1870. In many countries the standards for fatigue have been revised and based on the stress range concept. The work of ECCS committee T6 led to the harmonization of fatigue design concepts, resulting in the standards concerning fatigue of Eurocode 3.

Now there are the tools to evaluate the welded bridges of the past, which may have bad details that not have been recognized as such at the time of the design.
4. Comparison of stresses in steel railway bridges due to a standard design loading

To compare the load carrying capacity of old bridges designed according to different design loadings and different allowable stresses, it is convenient to calculate the stresses in these bridges under the standard design loading of the UIC (Union International de Chemin de Fer), shown in figure 2, as a reference loading.

To establish the action of the various design loadings, these loadings are transformed into equivalent distributed loads which cause the same maximum bending moments as the design loadings.

Figure 2 also shows the equivalent distributed loads of the applied loadings in the Netherlands between 1877 and 1917.

For comparison the equivalent distributed load of the UIC loading is given in this figure.

Figure 3 provides the allowable stresses of the past.

When the design load and the used allowable stress of an existing bridge are known, and also the dead load, it is possible to calculate that part of the stress representing the live load in the past time.

Now it is possible to calculate the maximum stress under the UIC-loading and dead load.

Figure 4 provides the results.

Rating bridges to fatigue the stress range is the governing parameter. That is why further the stress range under the UIC loading \( \Delta \sigma_{UIC} \) is calculated from the ratio between the equivalent distributed load of the UIC loading and that of the past time.

The calculations are carried out taking into account the impact factor \( \phi \) of the UIC-loading and the impact factor of the design loading of the past, provided this has been applied at the time.

Fig. 5 shows the results of the calculated stresses \( \Delta \sigma_{UIC} \) in bridges constructed between 1877 and 1917.

When the stresses \( \Delta \sigma_{UIC} \) are known, there is a possibility to have a relationship to the actual stress ranges due to real trains. The actual stress ranges can be expressed in a proportion of the stress range \( \Delta \sigma_{UIC} \).

This opens the way to investigate the fatigue problem.

5. Relationship between the value \( \Delta \sigma_{UIC} \) and the fatigue life of railway bridges under a given traffic

Knowing the composition of a traffic, stress-time histories can be calculated by running the trains of that traffic over static influence-lines.

Instead of the impact factor a so called fatigue-coefficient is used to represent the dynamic influence.

It has been shown from measurements that the application of the usual impact factor led to results which are too pessimistic while the impact factor covers maximum values of the dynamic effect. In reality there is a scatter of stresses which have to be considered.

A fatigue-coefficient being an average of the impact factor seems to be more realistic.

Using the rainflow counting method the stress-time histories are transformed into a number of stress cycles.

After that the evaluated stress ranges of the different trains are expressed in a ratio to the stress range \( \Delta \sigma_{UIC} \).
Doing this for several spans spectra of stress ranges can be composed for each span.
To evaluate the fatigue life the cumulative damage hypothesis of Palmgren-Miner is applied, using the fatigue strength curves of the ECCS.
The fatigue life calculations are carried out with real loads of a standard traffic with typical trains. This traffic is also used for new bridges.
Knowing the spectra with stress ranges expressed in a ratio to $\Delta \sigma_{\text{UIC}}$, the fatigue life can be calculated for bridges of several spans and a chosen value $\Delta \sigma_{\text{UIC}}$.
Doing this for several values of the stress range $\Delta \sigma_{\text{UIC}}$ the limiting values $\Delta \sigma_{\text{UIC}}$ can be presented as a function of the fatigue life.
Figure 6 provides the course of the procedure using an arbitrary SN-curve.
While all fatigue strength curves of the ECCS are similar, having the same slope and the knee at the same number of cycles $N = 5 \times 10^6$, a simplification in the calculations can be achieved by transforming the stress range $\Delta \sigma_{\text{UIC}}$ into a dimensionless value, dividing $\Delta \sigma_{\text{UIC}}$ by the value $2.10^6$ of the considered SN-curve.
Now factors, $k$, applicable for all SN-curves can be evaluated to define the limiting stress range $\Delta \sigma_{\text{UIC}}$ for an assumed life and the standard traffic.

$$\Delta \sigma_{\text{UIC}} \text{ (Lim)} = \frac{\Delta \sigma_{\text{UIC}}}{k \cdot \gamma}$$

(1)

Figure 7 shows the derivation and gives a set of factors $k$ for an assumed fatigue life of 50 years, respectively 100 years, for the mentioned standard traffic.

6. Evaluation fatigue life of riveted railway bridges

To evaluate the fatigue life of bridges it is necessary to know the changes in traffic during the life of the structure. In many cases exact data about that are not available.
To have an idea of the affection to fatigue, the fatigue life calculations are carried out using the spectra with the stress ranges of the standard traffic, assuming that this traffic was present during the whole life of the bridge. The limiting stress ranges $\Delta \sigma_{\text{UIC}}$ are compared with the static reference value $\Delta \sigma_{\text{UIC}}$ due to the UIC loading scheme.
To prove the sensitivity of the choice of the SN-curves the calculations of the limiting stress ranges $\Delta \sigma_{\text{UIC}}$ are carried out for the ECCS curves 71 and 90. A safety factor of $\gamma = 1$ is used.
The figures 8 and 9 provide the results of the comparison.
The standard traffic has an annual tonnage of $26.10^6$.
Calculating the limiting stress ranges $\Delta \sigma_{\text{UIC}}$ not only for a fatigue life of 100 years, but also for a life of 50 years, the influence of a traffic with an annual tonnage of $13.10^6$ during 100 years can be considered, assuming that for that traffic the ratio between the number of passenger trains and freight trains is the same as that of the standard traffic.
It can be seen from figure 8 that for the SN-curve EGC71 dependent on the annual tonnage wrought iron bridges constructed according to the specifications of 1877 with spans until about 17 m or 22 m are sensitive to fatigue after a life of 100 years for the considered traffic. Figure 9 shows limits of 7 m or 18 m for steel bridges constructed in 1903 and limits of 6 m or 9 m for bridges of 1917 comparing the curves EGC71 and 90.

In the case of smaller spans the calculated limiting stress ranges become so small that the actual stress ranges should have values below the fatigue strength. Taking into account that when all actual stress ranges are below the value of the stress range at $N = 5 \times 10^6$ of the SN-curve, no fatigue damage occurs, the limiting stress ranges $\Delta \sigma_{UIC}$ are cut off corresponding to the limiting values of the actual stress ranges.

Cross girders are bridge members seriously subjected to fatigue. Figure 10 shows the comparison between the static reference value $\Delta \sigma_{UIC}$ and the limiting stress range $\Delta \sigma_{UIC}$ with respect to fatigue of some cross girders.

It is obvious that cross girders made of steel St 52 give bad results from a point of view of fatigue.

Assuming that the traffic of the past is not heavier than the used standard traffic, this procedure provides a first step to rate bridges concerning fatigue.

6.1 Influence composition of the traffic

The composition of a traffic may have a great influence on the fatigue effect. The contribution to the fatigue damage caused by a heavy freight train is different from that caused by a passenger train. The extent of damage due to the several kinds of trains also depends on the length of the span. In figure 11 the extent of the damage due to different trains is represented for the used standard traffic. The proportion of damage due to the different trains has been calculated for a limiting stress range $\Delta \sigma_{UIC} = 140 \, N/mm^2$ using a SN-curve with a slope $m = 3$ without a knee. The composition of the traffic concerning the percentage of passenger trains and freight trains is presented for the tonnage of trains. The number of each kind of trains is also given. The freight trains are distinguished between mixed freight trains and heavy freight trains (blocktrains).

It can be seen from the figure that for the smaller spans the freight trains, especially the blocktrains, have a great part in damage. In this region the axles are dominating. In the case of longer spans a run of one train causes only one cycle and the value of the total weight of the train dominates. That is why the heavy freight trains also have a great part in damage in this area. For spans of about 15-20 m the passenger trains are most damaging. In general freight trains are more damaging than passenger trains, but for the mentioned spans the distance between the bogies of the passenger trains is such that each wagon causes one cycle. Taking into account the great number of passenger trains it is clear why they are so damaging in that area.
6.2 Relationship between fatigue damage and carried total tonnage of train traffic

After the first general rating of the sensitivity of bridges to fatigue further investigation has to be carried out to evaluate the remaining fatigue life of the structure. For that the knowledge about the load history and future traffic is necessary.

However in many cases exact data about past traffic fail. In general railway administrations but have statistics about the annual tonnage transported over the years.

When train traffic is distinguished between passenger and freight trains, the parameter fatigue damage per ton for the passenger trains and for the freight trains have each practically constant values for a considered span and an assumed stress range \( \Delta \sigma_{\text{UIC}} \), irrespective of the total tonnage of the traffic.

Assuming that the proportion of the passenger and freight trains of the past traffic is more or less the same as that of the present used standard traffic, a further approximation of the damage due to the past traffic is possible using the parameter of damage per ton.

Using the relationship between the dimensionless value \( \Delta \sigma_{\text{UIC}} / \Delta \sigma_{2.10^6} \) and the fatigue life presented in years (fig. 7), the relationship between \( \Delta \sigma_{\text{UIC}} / \Delta \sigma_{2.10^6} \) and the total carried tonnage can be defined by multiplying the years by the annual tonnage (26\( \times 10^6 \)) of the standard traffic.

Figure 12 shows the results.

Now it is possible to define the total tonnage that can be carried during the fatigue life when the calculated value \( \Delta \sigma_{\text{UIC}} \) and the value \( \Delta \sigma_{2.10^6} \) of a chosen SN-curve are known.

On the other hand, for a given transported tonnage and a given value of \( \Delta \sigma_{\text{UIC}} \) an estimation can be made of the fatigue damage.

An example of this assessment is given in figure 13 for a riveted railway bridge with a span of 10 m and a calculated stress range \( \Delta \sigma_{\text{UIC}} = 125 \text{ N/mm}^2 \).

The total transported tonnage has an amount of 1000\( \times 10^6 \) tons.

A comparison is made between the application of the SN-curves ECCS 71 and 90.

This example shows that the choice of the SN-curve has a great effect on the estimation of the damage.

In general the presentation of the damage as a Palmgren-Miner summation is a more convenient information than the fatigue life expressed in years.

7. Measurements of the live load stresses

It has been shown from measurements that generally the measured stresses are lower than the calculated stresses.

An important feature in connection with this phenomenon is the simplified theoretical stress analysis.

In most cases a simple calculation according to common engineering practice is used.

However in reality more elements of the structural system act together. Also the distribution of the loads may be more than assumed.

In other cases fixed-ended situations of beams, e.g. cross girders, are disregarded, and lead to higher calculated stresses.
With improved models of the structural systems the differences become smaller.

Another explanation of the stress differences is a conservative assumption of the traffic loads.

On the other hand there may be situations where the measured stresses are higher than the calculated stresses. This can occur at regions near the end of bridges due to heavier impact caused by imperfections of the track at the abutments.

When fatigue life calculations result in a negative life, measurements can give a decisive answer whether the assumptions have been too conservative or not.

8. Determination of inspection intervals

Even when conservative assumptions are corrected by results of stress measurements there may be situations of a calculated negative remaining life while in reality in a bridge no cracks are found. This situation especially can occur when little is known about previous loading and safe assumptions have to be made for that. In such cases bridges have to be inspected frequently awaiting a decision of possible replacement.

Starting from an actual situation that no cracks have been found, but assuming a certain crack size in the most critical bridge element, the use of fracture mechanics may be helpful to define the length of an inspection interval in which the assumed crack does not grow to a critical length.

In order to determine a safe inspection interval one needs information about crack propagation.

The crack growth rate \( \frac{da}{dN} \) is described by the Paris power law.

\[
\frac{da}{dN} = C \cdot \Delta K^m
\]  

(2)

where \( \Delta K \) is the range of the stress intensity factor. \( C \) and \( m \) are constants.

To define the crack growth rate, fatigue crack growth tests were carried out with center-cracked specimens of early steel.

Figure 14 shows the results presented as a relationship between the crack growth rate, \( \frac{da}{dN} \), and the range of the stress intensity factor \( \Delta K \).

Adopting for the constant \( m \) the value \( m = 3 \), for the constant \( C \) was found \( C = 4 \cdot 10^{-13} \) for the upper bound, and \( C = 1 \cdot 10^{-13} \) for the lower bound.

Fatigue crack growth tests were also carried out with specimens made of wrought iron. The results are shown in figure 15.

It can be seen from this figure that there is a great scatter. At lower values of \( \Delta K \) the crack growth is slower than that of early steel.

At higher values of \( \Delta K \) the maximum magnitudes of the crack growth rate are close to the upper bound of early steel.
8.1 Inspection intervals riveted bridges

Fatigue cracks in riveted structures generally emanate from rivet holes at highly stressed locations. In the case of sections deteriorated by corrosion, cracks can develop at edges with reduced area. The conservative assumption is made that, having a built up member, all parts simultaneous start to crack at the same location at both sides of a hole, and redundancy is not taken into account. Cracks at rivet holes nearest the edge of members were assumed to be most severe.

Determining inspection intervals it is important to know how long it will take the crack to grow from the minimum detectable size to the critical size. Experience [3] has indicated that performing a visual inspection with the aid of a magnifying glass and good illumination, cracks can be detected when they appear about 10 mm beyond the rivet head. Such a crack means a size of about a = 26 mm.

To evaluate an inspection interval a crack propagation of a_i = 28 mm to a_j = 33 mm is assumed. That includes a possible crack extension during the inspection interval of 5 mm.

Fracture mechanics calculations have shown that dependent on the width of the components and the edge distance of the rivets, crack sizes of a = 35 mm to a = 40 mm become critical.

It has to be pointed out that when cracks of such sizes in reality are found in primary members, repair or replacement of a bridge has to be considered.

Numerical integration of the crack growth rate description between the two crack sizes results into a SN-curve for a crack, propagating from a_i = 28 mm to a_j = 33 mm. For the constant C the value C = 4.10^{-13} is used.

\[
N = \int_{a_i}^{a_j} \frac{da}{C \cdot \Delta K^3} \tag{3}
\]

where \( \Delta K = F(a) \cdot \Delta G \cdot \sqrt{\pi a} \)

\( F(a) \) is a geometric correction factor.

For several configurations SN-curves are presented in figure 16, indicated by the value \( \Delta G \) (gross section) at \( N = 2.10^6 \) cycles and a slope \( m = 3 \).

A fatigue limit is disregarded.

To determine the fatigue crack propagating life, growing the crack between the two crack sizes, a representative stress spectrum with variable amplitude and a number of cycles \( n_i \), is transformed into a spectrum with an equivalent stress range \( \Delta G_e \) of constant amplitude and the same number of cycles \( n_e \), causing the same damage.

\[
\Delta G_e = \left[ \frac{\sum_{i=1}^{n_i} \cdot \Delta G_i^3}{n_e} \right]^{1/3} \tag{4}
\]
Using the SN-curve for the crack extension of 5 mm and the equivalent stress range $\Delta C_e$ (gross section) the propagating life $N$ can be computed.

Now the inspection interval $\Delta T_i$ has to be estimated as:

$$\Delta T_i = \frac{N}{n_s}$$ (5)

Only cracks visible from outside can be detected by a visual inspection.
At splices internal cracks cannot be detected visually before reaching the edge and then they are unstable.
In such cases it is necessary to apply radiographic inspection methods.

**Numerical example (Figure 17)**

Suppose a riveted railway bridge with a span of 10 m has been in service for about 85 years.
Recalculation with the UIC-loading included dynamic increment results in a stress range $\Delta C_{UIC} = 125 \text{ N/mm}^2$ (net section).
The measured stress range spectrum yields to an equivalent stress range $\Delta C_e = 25 \text{ N/mm}^2$ (gross section) and a number of cycles $n_s = 8.9.10^5$ per year.

Using the evaluated SN-curve with $\Delta C = 15 \text{ N/mm}^2$ at $N = 2.10^6$ cycles and $m = 3$, the propagating life $N$ for $\Delta C_e = 25 \text{ N/mm}^2$ is computed at $N = 4.33.10^5$ cycles.

The inspection interval is estimated at:

$$\Delta T_i = \frac{N}{n_s} = \frac{4.33.10^5}{8.9.10^5} \approx 0.5 \text{ year}$$

**8.2 Inspection intervals welded bridges**

The first generation welded bridges were modified riveted ones. In a number of cases sharp edges and discontinuities are present. At these locations fatigue cracks can occur.
Examples are gusset plates welded to the flange tip or to the web of a beam, and cover plates.
To evaluate an inspection interval it is assumed that the minimum size of a detectable crack has a magnitude of 10 mm.
Adopting this initial crack size and using the method of fracture mechanics the remaining fatigue life of the governing weld details can be calculated for a given equivalent stress range.
In this way the region of unstable crack extension can be defined. Then the results are extended to estimate the inspection interval.

**Numerical example (Figure 18)**

From a welded railway bridge with a span of 10 m the propagation of a crack at the weld toe of a gusset plate attached to the flange tip of the main girder is considered.
An initial crack $a_i = 10 \text{ mm}$ is assumed.
Using the expression of Eq (3) the fatigue crack propagation life can be calculated for a given equivalent stress range. Approximately the correction factor $F(a)$ is estimated to be a product of the correction factor for edge cracks, $F_s'$, and the geometry correction factor $F_g$ representing the influence of the global stress concentration due to the gusset plate. [4]

$$F(a) = F_s' \cdot F_g$$

where $F_s' = 1,12$ and $F_g = 1,5$

Using the spectrum of a railway bridge with a span of 10 m, having an equivalent stress range of $\Delta \sigma_e = 25 \text{ N/mm}^2$ and a number of cycles $n_s = 8,9 \cdot 10^5$ per year, for the constant $C = 4.10^{-13}$, a critical crack size of $a = 70 \text{ mm}$ is calculated after $N = 1,95 \cdot 10^6$ cycles. That means a period of 2 years.

Assuming an inspection interval of 1 year the extension of an initial crack $a_i = 10 \text{ mm}$ during the first period has an magnitude of 8 mm. The next period the crack extends to the critical size $a = 70 \text{ mm}$.

9. Brittle fracture

To investigate the risk of brittle fracture of existing steel structures it is necessary to have an estimate of the critical values of the stress intensity factors concerning the used material. Modern steels have guaranteed fracture toughness, old steels however may have bad properties due to the process of manufacturing or originating from the chemical composition.

9.1 Risk brittle fracture of riveted bridges

9.1.1 Mild steel

Assuming a critical stress intensity factor $K_c = 3000 \text{ N/mm}^{3/2}$ for mild steel, the adopted limitation of crack extension to define an inspection interval, being a propagation $a_i = 28 \text{ mm}$ to $a_j = 33 \text{ mm}$, in general will not lead to a risk of brittle fracture while from a point of view of static strength the maximum stress shall not be higher than $\sigma = 140 \text{ N/mm}^2$ (gross section).

Assuming an average correction factor $F(a) = 1,6$ for a crack size $a_j = 33 \text{ mm}$ the stress intensity factor has a magnitude of about: $K' = 1,6 \cdot 140 \sqrt{33} = 2300 \text{ N/mm}^{3/2}$.

However some old mild steels are sensitive to ageing. Especially steels with a high content of phosphorus, sulphur and nitrogen. An example of such a steel is Thomas-steel.

<table>
<thead>
<tr>
<th>Steel</th>
<th>Chemical composition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C</td>
</tr>
<tr>
<td>Thomas-steel</td>
<td>0,04</td>
</tr>
<tr>
<td></td>
<td>0,06</td>
</tr>
</tbody>
</table>
In the boundary of rivet holes ageing can occur due to riveting. Especially when during driving the rivets, by bad workmanship, the snap tool has been penetrated into the parent material of the structure. In this case a small fatigue crack emanating from a rivet hole can initiate a brittle crack. Dependent on the magnitude of the maximum stress the brittle crack will stop and then continues as a fatigue crack, or the initial crack will not stop and propagates as a total brittle through crack. From the results of fatigue crack growth tests it was found that more or less at a level of the maximum stress: $\sigma_{\text{max}} = 75 \text{ N/mm}^2$ (gross section) at a temperature of $-20^\circ\text{C}$ this transition can appear. That means that in cases of severe local ageing due to penetration of the snap tool total brittle fracture of a component already can occur when the stress intensity factor reaches a magnitude of about $K = 500 \text{ N/mm}^{3/2}$. Figure 19 shows the results of crack growth tests with center-cracked specimens. The crack-growth tests were carried out with a frequency of 10 Hz. For the used stress $\sigma = 75 \text{ N/mm}^2$ this corresponds with a strain rate $\frac{\Delta e}{\Delta t} = 7 \times 10^{-3} \text{ sec}^{-1}$. The strain rate of a railway bridge with a span of 10 m varies between $2 \times 10^{-3}$ and $3 \times 10^{-3} \text{ sec}^{-1}$. The evaluated critical stress intensity factor of $K = 500 \text{ N/mm}^{3/2}$ therefore can be seen as a dynamic stress intensity factor.

It has to be pointed out that defining the inspection interval the assumption had been made that all parts of a built up member simultaneous crack at both sides of a rivet hole. This is a very unfavourable assumption.

9.1.2 Wrought iron

Until now there is only little information available about the magnitude of the critical stress intensity factor of wrought iron. The results of the fatigue growth tests mentioned previously have shown that the critical stress intensity has a magnitude of at least $K = 1200 \text{ N/mm}^{3/2}$. Recently investigations about this feature have been started at ICOM in Switzerland [5].

9.2 Risk brittle fracture of welded bridges

To rate existing welded bridges concerning the risk of brittle fracture it is necessary to know the mechanical properties and chemical composition of the used steel. About 1935 electric arc-welding was introduced in bridge building. Between 1935 and 1940 some welded bridges collapsed, caused by brittle fracture. It was shown that rimmed steels are most prone to brittle fracture. The application of killed steel and steel with a low transition temperature do diminish the risk of brittle fracture.

When welded bridges have been constructed of a steel not guaranteed to have a satisfactory notch toughness there may be a risk of brittle fracture at the time that small fatigue cracks occur. These sharp and hardly detectable cracks suddenly can cause a brittle crack propagating very rapidly through the whole structure.
10. Distortion induced fatigue cracking

Many times cracks occur at locations where simplified stress analyses do not indicate that. In those situations cracks mostly can be explained by the phenomenon of induced distortion. This may be out of plane bending or induced displacement. Figure 20 provides some examples. In general these cracks are not directly detrimental to the serviceability of the bridge when they are parallel to the direction of the primary stress. However to avoid crack growth perpendicular to the primary stress retrofitting in time is necessary. It has to be pointed out that rating bridges to fatigue it is very important to use models of calculation as real as possible to obtain stresses most reliable to judge.

11. Influence of structural details

Sometimes it may be necessary to investigate the stress distribution at the location of a structural detail itself while this distribution can differ from that what is assumed in common engineering practice.

11.1 Splices

Figure 21a shows an example of a symmetric splice in which the pattern of the rivets causes a non-uniform distribution of the stress. In figure 21b the influence of asymmetric splicing is shown.

11.2 Direct connection of web members to the chords of truss bridges

Many times, especially in old wrought iron lattice and double truss bridges, gusset plates fail. In this situation the chord members have not only to carry the primary forces, but have to act as gusset plate as well. Failing of gusset plates causes considerable local stress raising in the chords.

To evaluate the remaining life of an old wrought iron truss bridge three scale models were made of this connection and proved in the Stevin Laboratory of the TU-Delft. The results of the fatigue tests of the three specimen did not differ much. Cracks occurred after about \( N = 2 \times 10^6 \) cycles at a stress range \( \Delta \sigma = 125 \text{ N/mm}^2 \) in the highest stressed region.

As a result of these tests later on some rivets at highly stressed locations in the tension chords were replaced by injected HSFG-bolts in order to reduce the peak stress near to the rivet.

Figure 22 shows the measured stress distribution in the chord at the location of the direct attachment of a diagonal and a standard.

11.3 Coped beams

Connecting stringers to cross girders the flanges of the stringers frequently are coped. The coped beams may have one or both flanges removed.

Due to the discontinuity of the flange and the web at the coped corner high stress concentration can occur at this location, depending on the magnitude of the radius between the flange and the web.

Many times cracks were observed at the corner of the coped beams. Not only cracks occur emanating from copes at the tension side but, due to the residual stresses in the beams, cracks can also be initiated under cyclic compression.

The primary cause of fatigue cracking at the copes is the transmitting of the shear force into the web of the stringer coupled with the stress concentration due to the geometry of the cope. This results in a high stress near the edge of the cope. Even when the reduction of the bending resistance of the stringers due to the copes is compensated by connection plates at the supports situated at the cross girders, cracks can occur in the same way.

The magnitude of the corner radius of the cope has a predominant influence with regard to fatigue. Small roundings or imperfections at the cope do reduce the fatigue life very strongly. Taking into account the situation that coped beams are very sensitive to fatigue cracking it is necessary to inspect these members frequently, especially in the case of change in traffic to heavier loads.

To have an idea of the extension of a crack overlooked eventually during an inspection, a crack size of 10 mm is assumed and the crack propagation is calculated using fracture mechanics.

The crack extension is roughly modelled by considering the cope as an initial deep edge crack. In general cracks at copes grow under an angle of 45°. That is why modelling the problem the end of the coped beam is considered to be a part of a plate turned 45° with respect to the center line of the beam (fig. 23).

The width of the plate with edge crack is estimated at:

\[ w = (h - c) \cdot \sqrt{2} + b/\sqrt{2} \quad (7) \]

where:
- \( h \) = height of the beam
- \( b \) = cope length
- \( c \) = cope deepness

It is assumed that the considered plate with an edge crack is loaded by the principal stress at the end of the beam, being the shear stress range \( \Delta \tau \).

The stress intensity factor is defined as

\[ \Delta K = F_s \cdot \Delta \tau \cdot \sqrt{\pi a} \quad (8) \]

Where \( F_s \) is the correction factor for edge cracks.

This assessment is an approximative interpretation of the results of fatigue tests with coped short rolled beams. Figure 23 provides the results of the crack growth of one of the specimens.
Numerical example (Figure 24)

Suppose a railway bridge with a floor-beam-stringer system. The stringers with a span of 4 m were fabricated from rolled beams DIN 380 with coped flanges. The calculated shear stress range due to the UIC-loading scheme yields to \( \Delta \tau_{\text{UIC}} = 92 \, \text{N/mm}^2 \).

The equivalent stress range due to the actual train traffic is estimated at \( \Delta \tau_{\text{e}} = 18.5 \, \text{N/mm}^2 \) and a number of cycles \( n_s = 2.42 \times 10^6 \) per year.

The magnitude of the assumed edge crack taking into account an initial crack of 10 mm from the edge of the cope is:

\[
a = \sqrt{\frac{185}{2}} + 10 = 140 \, \text{mm}
\]

Using in equation (3) a constant \( C = 3 \times 10^{-12} \) the calculated critical crack size has a magnitude of 270 mm after about \( N = 9 \times 10^5 \) cycles. That means a crack size \( a = 130 \) mm from the edge of the cope after a period of:

\[
\Delta T = \frac{9 \times 10^5}{2.42 \times 10^6} = 0.4 \, \text{year}
\]

This example shows a very rapid extension of the crack after an initial crack of 10 mm from the edge of the cope.

12. Concluding Remarks

The major problem evaluating bridges concerning fatigue life prediction resides in the fact that in many cases exact data about past load histories fail. That is why it is difficult to judge if there is a safety margin or not in such cases. A rough estimation of the fatigue life can be achieved calculating the fatigue life using standard spectra based on actual traffic. For these reasons field inspection is essential to detect fatigue damage in an early stage. When calculations tend to a risk of fatigue damage but no cracks are observed, or in cases of uncertainty about fatigue life fracture mechanics methods may be helpful to determine inspection intervals based on the assumption of the presence of cracks with the minimum detectable size.

Acknowledgment

The author wishes to express his gratitude to Professor J. de Back for the many years of fruitful cooperation.
References

[1] UIC code leaflet 702-0.

Statistical distribution of axle loads and stresses in railway bridges.

[3] Brühwiler, E, Hirt, M.A.
Das Ermüdungsverhalten genieteter Brückenbauteile.

Fatigue Life Estimation Using Fracture Mechanics.
IABSE Colloquim Lausanne 1982.

Bewertung der Spontanbruchgefahr angerissener Brückenbauteile aus Schweisseisen.
Der Stahlbau Vol 58, no 1, 1989.

Figure 1 SN-curves of riveted connections.
Influences affecting the fatigue strength.

Figure 2 Comparison of equivalent distributed design loadings.
Figure 3 Allowable stresses of the past.

Figure 4 Maximum stress due to UIC-loading and dead load in existing railway bridges.

Figure 5 Stress range due to UIC-loading in existing railway bridges.
Static influence line bending moment midspan

Train with fatigue factor:
\[ S_f = 1 + 0.5(\Delta E + 0.5 \Delta \epsilon) \]

Stress-time history

Rainflow counting method

Cumulative stress-spectrum

Palmgren-Miner rule

Allowable stress as a function of life

Figure 6 Procedure evaluation fatigue life.

Figure 7 Derivation of factors k to define the limiting stress range \( \Delta \sigma_{UIC} \).
Comparison of stress ranges $\Delta \sigma_{UIC}$ in old wrought iron bridges with limiting stress ranges $\Delta \sigma_{UIC}$ concerning fatigue.

Comparison of stress ranges $\Delta \sigma_{UIC}$ in existing steel railway bridges with limiting stress ranges $\Delta \sigma_{UIC}$ concerning fatigue.

Comparison of stress ranges $\Delta \sigma_{UIC}$ in existing steel cross girders with limiting stress ranges $\Delta \sigma_{UIC}$ concerning fatigue.

Relationship $\Delta G_{UIC}/\Delta G_{2.10^6}$ and carried total tonnage.

$\Delta G_{UIC}/\Delta G_{2.10^6}$ and carried total tonnage.
Figure 11 Rate of damage due to different kinds of trains of a standard traffic.

Figure 13 Numerical example of the assessment to evaluate the fatigue damage due to the carried total tonnage.

Fig 14 Relationship between crack growth rate da/dN and ΔK of early mild steel.

Fig 15 Relationship between crack growth rate da/dN and ΔK of wrought iron.
Figure 16  SN-curves crack growth $a_i = 28$ to $a_i = 33$ mm for several configurations of riveted structures (mild steel).

Numerical example

Riveted railway bridge

$\Delta G_{\text{net}} = 125 \text{ N/mm}^2$ (net section)

$\Delta G_{\text{gross}} = 100 \text{ N/mm}^2$ (gross section)

$\Delta G_e = 25 \text{ N/mm}^2$

$n_e = 8.9 \times 10^5$ cycles/\(\gamma\)

measured stress spectrum (1 year)

$\Delta T_i = N = 4.33 \times 10^5$

$e = 0.5$ year

Figure 17  Example determination inspection interval of a riveted railway bridge.

Numerical example

Welded Railway bridge

$\Delta G_{\text{net}} = 100 \text{ N/mm}^2$ (net section)

$\Delta G_e = 25 \text{ N/mm}^2$

measured stress spectrum (1 year)

$n_e = 8.9 \times 10^5$ cycles/year

Figure 18  Example crack extension in a gusset plate of a welded railway bridge, assuming an initial crack of 10 mm.
Figure 19  Investigation brittle fracture of Thomas steel.  
Influence penetration of the snap tool into parent material.

Figure 20  Examples of distortion induced fatigue cracking.

Fig 21a  Stress distribution in  
riveted symmetric splice.  

Fig 21b  Stress distribution in  
riveted asymmetric splice.
Figure 22 Tests on scale models, direct connection of web members to the chord of a truss bridge.

Figure 23 Modelling copes in beams for fracture mechanics calculations. Fatigue crack growth tests with coped beams.

Figure 24 Example extension of an initial crack of 10 mm in a coped stringer of a railway bridge.
The special requirements of an aluminium fatigue design code are discussed. Existing codes and recent data evaluations are presented. Current research work of the author is presented in context with international aspirations to establish a commonly accepted recommendation. The basis is a comprehensive data collection with more than 15,000 data points. Preliminary results are presented and discussed, especially the effects of wall thickness and the R-value. The fields of future research requirements are outlined.

1. Introduction

Aluminium is the most widely used structural material in modern technology second to steel. Modern alloys produced by advanced technologies offer an ample spectrum of material properties, product shapes and joining methods of which the designer can make use for a competitive structure.

The main advantages of modern aluminium alloys are the light weight, an indefinite variety of product shapes by extrusion, good static strength of the material, corrosion resistance and the easy handling and joining in fabrication.

In opposition to this list of advantages there are some minor disadvantages to be considered and to be overcome by a good and advanced design. These are the buckling properties of a component which are a direct consequence of the modulus of elasticity. It is about one third of the value of steel. Secondly the lower resistance against fatigue, especially at welded components.

The buckling resistance is easily improved by the addition of adequate stiffeners or by the selection of appropriate sections, which can be produced by extrusion. As to the fatigue behaviour, a more detailed consideration is necessary. Design codes or recommendations should not just mirror those established for steel /1/. There, the material is so inexpensive that the economical optimum is more associated with simplicity in design and in analysis by code. For aluminium a variety of secondary (in terms of steel fatigue design) effects should be incorporated into the codes in order to maintain the basic advantages of the material without missing the requirements in respect to fatigue.

The well known difference in fatigue behaviour between small scale specimens and large scale thick walled components may be reasonably neglected at steel structures. The higher price of the material at aluminium might be prohibitive, if this procedure were directly trans-
ferred to an aluminium code. In other terms, it is highly desirable to give special consideration to two effects:

a) the wall thickness effect, which is active in the same way as at steel

b) the effect of residual stress and the R-value, which should be considered in more detail compared with steel

A modern fatigue design code for welded aluminium structures should describe the small scale components, which behave like laboratory specimens, and large scale components with thick walls and residual stresses as well. Well behaving structures for years in service should give the guidelines and it is intolerable to wreck them down by analysis using a new code.

2. Existing Codes

The major industrialized countries have developed fatigue design codes for aluminium structures. A recent overview /2/ shows that the results differ widely for certain cases of application. Table 1 gives an oversight of the most known codes and the aluminium alloys covered.

<table>
<thead>
<tr>
<th>Fatigue Standard</th>
<th>Country</th>
<th>AA alloy group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2000 3000 5000 6000 7000</td>
</tr>
<tr>
<td>CP 118</td>
<td>Great Britain</td>
<td>+ - + + + +</td>
</tr>
<tr>
<td>NS 3471</td>
<td>Norway</td>
<td>+ - + + + -</td>
</tr>
<tr>
<td>UNT 1985</td>
<td>Italy</td>
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<tr>
<td>SVR-norm</td>
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<td>ASCE 3341</td>
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<td>AS 1664(79)</td>
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</tr>
<tr>
<td>CAN 3-S157-M83</td>
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</tr>
<tr>
<td>Nor.Petr. 16/77</td>
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</tr>
</tbody>
</table>

Table 1: Fatigue design standards and alloys covered /2/

The table shows that the codes are usually confined to the mostly used types of alloys, i.e. the 5000, the 6000 and the 7000 type alloys. The numbers refer to the numbering system of the International Aluminium Association. These are in plain wording the AlMg alloys, the AlMgSi alloys for extrusions and the AlZnMg alloys for high strength welded structures.

The structural details of different types of welded joints, which are covered in the different codes are basically the same as in steel.

Entering different codes and calculating a special design problem a large variation of results is found, which necessarily must have its origin in the difference of the codes and not in different behaviour of the material according to the codes. An example will illustrate these differences: Given a tensile bar with a transverse stiffener welded on by two fillet welds. The tensile bar is cyclically loaded by a stress spectrum, in one case by a Rayleigh spectrum, in the other case by a
linear spectrum which simulates ocean wave loads. The allowable cyclic stresses are given in Table 3.

<table>
<thead>
<tr>
<th>Fatigue Standard</th>
<th>Country</th>
<th>Effects considered</th>
<th>R-ratio</th>
<th>Thickness</th>
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<td>no</td>
<td></td>
</tr>
<tr>
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<td>USA</td>
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<td>no</td>
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</tr>
</tbody>
</table>

Table 2: Effects considered in different codes /2/

Regarding the numbers of Table 3, it could be concluded that any arbitrarily chosen number (within certain limits) could do the job. It is evident that further research activities are needed to throw more light on the fatigue behaviour of welded aluminium joints. The way to achieve that is clear. A commonly accepted data base is needed, accepted evaluation procedures should be performed and the white spots have to be determined in order to have special tests on special specimens or large scale components to fill these gaps. In a last step a code may be established which meets the requirements stated above.

<table>
<thead>
<tr>
<th>Fatigue Standard</th>
<th>Country</th>
<th>Stress spectrum R-rayleigh Linear</th>
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<td>AS 1664(79)</td>
<td>Australia</td>
<td>74 140</td>
</tr>
<tr>
<td>CAN 3-S157-M83</td>
<td>Canada</td>
<td>43 90</td>
</tr>
</tbody>
</table>

Table 3: Variation of allowable stresses in different codes for a given example problem /2/

3. Recent Results

One of the last major evaluations of aluminium fatigue data was carried out one decade earlier by Atzori et al. /3/. Existing data collections, mainly the CIDA collection, were taken as the basis of investigations. Almost all of the data referred to small scale laboratory specimens with a wall thickness ranging from 4 to 12 mm. A special evaluation method was applied using the well known uniform behaviour of welded joints in connection with the pattern recognition capability of the human eye. The results are listed in Table 4. The numbers in parentheses refer to a more conventional analysis using a simple log-linear regression analysis.
Table 4: Results of an evaluation of small scale specimens /3/

It is interesting to see that the values of table 4 are in a large extent proportional to those derived from steel specimens. Hence there have been proposals to take steel fatigue design codes and divide the stress ranges by three. This procedure could hold for a first engineering approximation. The problem is that secondary effects like R-ratio or wall thickness are not taken in account. Effects which have been generously neglected in steel for all types of structures because of the cheapness of the material. In the case of aluminium, some types of structures could be made more expensive and uneconomic than necessary.

An additional collection and evaluation of the fatigue behaviour of large scale components is needed. Additional full size test should be carried out in a more systematic way rather than merely evaluating cases of damaged components. Tests like this are under the way and first results are available /4/.

4. Tools for Research Activities

On establishing fatigue design recommendations or codes it is not sufficient to have test results of own laboratory tests or evaluations of cases of damaged components. The data always will be scarce and reflect a small random sample on behaviour of welds. A more comprehensive collection of data acquired and published elsewhere is necessary.
A powerful tool are modern computer based data bases for literature, e.g. METADEX (USA), WELDASEARCH (GB) and DS (F.R. Germany). All these literature data bases have been searched in 1987 using the following keywords:

As to general fatigue: Aluminium, fatigue, welded joint
As to crack propagation: Crack, fatigue, welded joint

4.1 Aluminium Fatigue Data

About 350 references could be produced by the literature search, of which 250 have been found to be of use. The original references and literature have been procured, filed and entered in a data base. The number of publications and the number of SN-curves per annum found by literature data base searching are visualized in figures 1 and 2.

Together with already existing data of unpublished sources, older references, CIDA collection and CAFDEE data not covered elsewhere, a database was established containing up to now roughly 1500 test series descriptions and 15,000 individual data points.

The data base is bilingual and is based on an open language thesaurus of significant key words. It is not a retrieval system. It is a relational data base system using the principles of the entity - relation architecture. So, four files have been established. In addition, different result files have been created in order to store the data of different evaluation procedures. These files are not an integer part of the data base and may be created according to the specific requirements of the evaluation procedure and presentation of results.

FILES OF THE DATA BASE

<table>
<thead>
<tr>
<th>Filename</th>
<th>Entity -&gt;</th>
<th>Relation -&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Litfile:</td>
<td>Literature references</td>
<td>Literature number</td>
</tr>
<tr>
<td>Testfile:</td>
<td>Description of test series</td>
<td>Test number</td>
</tr>
<tr>
<td></td>
<td>Test number</td>
<td>Literature number</td>
</tr>
<tr>
<td>Datafile:</td>
<td>Fatigue data of the individual specimen</td>
<td>Test number</td>
</tr>
<tr>
<td>Reffile:</td>
<td>Literature number</td>
<td>Test number</td>
</tr>
<tr>
<td></td>
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<td>Test number</td>
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<td>Test number</td>
</tr>
<tr>
<td>Res2file:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

More details and more background information on this database structure and requirements are given in ref. /5/. Practically the database is based on an IBM type MS-DOS computer with fixed disk and on DBASE III+. 
Graphics capabilities have been added.

It is felt that this structure of a fatigue data base is a valuable and powerful tool. It has become something like a standard for fatigue research application and is also used elsewhere in Germany. Figures 3 and 4 give an example of record structures of the literature file and the test description file. Figure 5 shows the graphic representation of the data of one test series.

As a final remark, it has to be accepted as a fact that how intricate the data base structure ever might be it is indispensable for the researcher to have the original literature readily available within his reach.

4.2 Crack Propagation Data

Besides the classical fatigue data of welded structures, the fracture mechanics crack propagation behaviour is of major interest in welded joints. In this field basic research activity is carried out on steel and not on aluminium. So the search for information and data was not confined to aluminium.

The literature database search returned about 850 references, of which 540 have been found to be useful and have been procured in original literature. Figure 6 shows the number of publications per annum retrieved from the literature data bases. A new database has been established, containing the bibliographic data, a summary in respect to the intentions of the research project and a list of separate keywords, by which the data entries can be retrieved.

The crack propagation data have been extracted and put into an additional file. It gives information about materials used, test conditions and the da-dN values. This file up to now contains the data of about 650 different da-dN curves. Unfortunately only 20 da-dN curves refer to aluminium welds. In contrast, aluminium base metal data are numerous. They are not included in the file. The data will be taken from elsewhere. So, at the moment studies have to go on using steel weld and aluminium base metal data and transferring results to aluminium welds.

5. Current Research Activities

The wide field of research topics in aluminium weld fatigue requires a broad collaboration between research institutes. Different effects have to be investigated like the effect of the R-ratio, of wall thickness, of alloy type and the difference in fatigue between small scale specimens and full scale components.

The different effects can be most easily studied at small scale specimens. The major part of data needed is already available in the data bases. Gaps of information may be closed by specific test series. If these effects were clearly understood in future, full scale component results will give guidelines for the design of codes fulfilling the requirements discussed above.

In the author's institute efforts are directed on the study of differ-
rent effects by evaluating fatigue data and by performing specific test series in order to fill some gaps. Full scale test are carried out elsewhere, e.g. see ref. /4/. Besides the fact that a good portion of large scale fatigue data is already incorporated in the databank, there still exist company owned data, of which it is hoped to have them available in the near future as well.

5.1 Study of Different Effects

For the study of the effect of thr R-ratio and the wall thickness a mathematical model of the SN curve has to be established. According to conventions and literature studies the following model was chosen:

\[ N = C \cdot F(R, a, t, b, \Delta S, m) \]

where

- \( N \): load cycles
- \( C \): constant for the structural detail
- \( R \): R-ratio
- \( a \): exponent for the R-ratio
- \( t \): wall thickness in mm
- \( b \): exponent for the wall thickness
- \( \Delta S \): stress range in N/mm²
- \( m \): exponent for the slope of the S-N curve

A preliminary evaluation was carried out for two types of structural detail: The one sided transverse stiffener on tensile bar welded on by fillet welds and the cruciform joint in tension welded by K-butt welds without root gap.

The data have been extracted from the data base and have been processed by a multiple stepwise regressional analysis. The exponent for the slope of the S-N curve "m" was taken as 4.00. This value seemed to be reasonable by previous studies. The stress range for 2 million cycles (class in IIW recommendations /1/) was calculated by a simple regression.

The mathematical model was rearranged to:

\[ \text{Class} = \text{const} \cdot (1 - R)^a / t^b \]

or

\[ \log(\text{Class}) = \log(\text{const}) + a \cdot \log(1 - R) - b \cdot \log(t) \]

The exponent a and b can be directly used to describe the effect of R-ratio and wall thickness. The multiple regression was performed and the results are tabulated in table 5.

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>R-ratio</th>
<th>Exponents</th>
<th>wall thckn.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>STDV</td>
<td>Prob.</td>
</tr>
<tr>
<td>Stiffener</td>
<td>.26</td>
<td>.15</td>
<td>4.9 %</td>
</tr>
<tr>
<td>Crucif. j.</td>
<td>.58</td>
<td>.26</td>
<td>1.5 %</td>
</tr>
</tbody>
</table>

Table 5: Evaluation on secondary effects at two structural details
The effect of the R-ratio is evident as expected for small scale specimens. The value differs strongly, but it could be within the limits of one standard deviation for each side. The effect of the wall thickness, as represented by "b", is more uniform. The effect is surprisingly high. The wall thicknesses ranged from 4 mm to 10 mm while the R-ratio ranged from -1 to +0.1.

At the moment no final conclusions can be drawn. The whole variety of structural details has to be studied. The results have to be applied on full scale components and checked for consistency. The investigations go on and a final document is expected in the near future.

5.2 Special Tests
Looking through the data base it was found that data on longitudinal stiffeners are scarce in comparison with other structural details. Unfortunately about one third of all data refers to butt welds. There have been no data for the new alloy for extrusions AlMgSi0.7. This alloy is of a special interest for the new German high speed railway. So a test program was started with 12 test series and 144 specimens of longitudinal attachments of various sizes, wall thicknesses and post weld treatments. 90 % of the tests are finished. A report will be published soon. Figure 7 gives a view on the test rig.

This is certainly not the last gap to be filled. It is planned to establish a list of required tests in order to fill the urgent gaps.

5.3 Improvement techniques
It was initiated by IIW bodies to focus on improvement techniques. About 90 S-N curves containing improvement techniques could be extracted from the data base. A first survey on existing literature and techniques will be published soon.

6. Future Requirements
For research purposes which are also connected with the establishing of the IIW recommendations on assessment of weld imperfections /7/ the literature search and the data collection of da-dN data has been carried out. Reviewing the references it was found, that on the one hand fatigue data of cracked components are reported, while on the other hand crack propagation measurements have been carried out on small scale standard specimens.

The connection between these data is the correction function in the "Paris" power law of crack propagation. This applies for the correction of the overall geometry which usually is different from an indefinite plate containing a center crack and to the correction factors Mk which take account of local geometries, e.g. a weld bead. Only a few formulae are available.

For practical application of fracture mechanics it is highly desirable to have the correction formula for each individual structural detail as given by fatigue codes. Here a wide field of future research activity is
open. It is intended to establish a list of these structural details with the formulae available and with the gaps still waiting for filling. Future activities of the author's institute will focus on this problems.

7. References

/1/ Hobbacher, A. et al.
Design Recommendations for Cyclic Loaded Welded Steel Structures.
IIW Doc. XIII-998-81/XV-494-81

/2/ Örjesäter, O.
Design Codes for Fatigue in Aluminium Structures.

/3/ Atzori B., Dattoma V.
Fatigue Strength of Welded Joints in Aluminium Alloys - A Basis for Italian Design Rules.
IIW Doc. XIII-1088-83/XV-550-83

/4/ Kosteas D.
Summarizing Results of the Aluminium Welded Beam Fatigue Programme.
IIW Doc. XIII-1240-87/XV-641-87

/5/ Hobbacher A.
Data Base Requirements for Research in Fatigue of Welded Aluminium Components.
4. Intern. Conf. on Aluminium Weldments (INALCO'88), Tokyo, Japan, April 1988

/6/ Hobbacher A.
Recommendations for Assessment of Weld Imperfections in Respect to Fatigue.
IIW Doc. XIII-1266/XV-659-88
Figure 1: Activity on aluminium weld fatigue research. Numbers of publications per year in literature data bases

Figure 2: Activity on aluminium weld fatigue research. Number of S-N curves published per year in literature data bases
Figure 3: Structure of the literature file in the aluminium weld fatigue data base (ALUFAT)

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<tr>
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</tr>
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<td>TEST X-REF :0015 0016 0017 0048 0049 0050</td>
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Figure 4: Structure of the test description file in the aluminium weld fatigue data base (ALUFAT)

<table>
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</tr>
<tr>
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<tr>
<td>FILLER :5183</td>
<td>: Rm : 324: Rp0.2 : 269: A5 : 12:</td>
<td></td>
</tr>
<tr>
<td>JOINT TYPE :Stumpfstoss V-Naht = butt joint V-groove</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TREATMENT :wie geschweissst = as welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PRODUCT :Blech = plate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DIMENSIONS SPECIM. WIDTH : THICKN. : 9.5: SECONDARY :</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WELD LENGTH : THICKN. : ANGLE :</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PROCESS :MIG 1 Lage je Seite = MIG single V (60-70 deg) one pass on each side</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LOADING :axial = axial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ENVIRONMENT:Luft = air</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MISCELLAN. :</td>
<td></td>
<td></td>
</tr>
<tr>
<td>R : 0.00: DATA POINTS : 9: MARK :</td>
<td></td>
<td></td>
</tr>
<tr>
<td>REMARKS :</td>
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</table>
Figure 5: Graphic presentation of the test points of a test series in the aluminium weld fatigue data base (ALUFAT)
Figure 6: Activity on crack propagation research in welded joints (steel and aluminium). Number of publications per year in literature data bases.
Figure 7: Test rig for small scale specimens and full size components showing investigations on longitudinal stiffeners.
INTERACTION BETWEEN RESEARCH, STANDARDISATION AND DESIGN

Manfred A. Hirt and Ian F.C. Smith
Swiss Federal Institute of Technology, Lausanne

Summary

Increasing competitiveness and complexity of modern construction projects are creating a growing need for interaction between research, standardisation and design. To date, most interaction has followed a traditional serial progression of the three; more comprehensive interaction is rarely achieved. In this paper, fatigue design is used to present some examples of interaction, and future work is proposed. Complete interaction is seen as a key to good and cost effective design.

1. Introduction

Development and progress in structural engineering originate from three activities: research, standardisation and design. It is interesting to examine cases where a single serial succession of these activities could be replaced by interaction between the three (Figure 1).

Generally, new structures are larger, lighter and more complex than traditional designs. In several areas of structural engineering, research is initiated when existing standards and codes become too restrictive for the design of such structures. However, design practice is often not influenced directly by research findings, but only through modifications to codes. A serial succession of design, research, standardisation, etc. is followed (Figure 1a).

For many years, research into the fatigue behaviour of civil engineering structures was not even considered to be essential or urgent since fatigue cracks do not occur at early stages of service life. Therefore, there can be long time period between design and discovery of fatigue cracking. Also, fatigue tests are expensive – particularly when investigating variable amplitude effects, since they require an important investment in testing equipment and personnel over a long period of time. Finally, the introduction of welding technologies transferred detailing responsibilities to steel fabricators. Unfortunately, this transfer was enhanced by an increase in use of computers. Among many designers, software for structural analysis has contributed to a declining interest in good detailing practice.

Until approximately thirty years ago, design codes of most countries accounted for fatigue of riveted structures with very simple clauses. Allowable stresses were defined to be a function of stress ratio. Later, these formulae were adapted to include yield strength of the base material. Over the years, additional clauses were introduced in order to accommodate butt-welded connections. These clauses were drafted using results from fatigue tests on small specimens.

Later, in spite of general acceptance of welded connections for bridge designs, little research had been undertaken for determining actual fatigue strength of these details. The AASHO Road Tests in the USA during the early sixties revealed that cover-plates, when welded to rolled steel beams, reduced fatigue strength [1]. It is interesting to note that bridges containing these beams had been included in the Road Tests at the last moment and were thought to be a complementary study only.
The unexpected results from these bridge tests, later confirmed by laboratory tests and, more importantly, by fatigue cracking of actual highway bridges [2], led to more research activity. More specifically, an important test program comprising about 350 beams, 3m long, was undertaken at Lehigh University [3]. More tests followed, in the USA as well as in Europe and Japan. Beams were tested in order to account for the effects of residual stresses. Also, an important effort was made in the field of off-shore structures [4] since fatigue cracking can be the critical limit state.

Introduction of the findings of these research programs into European design codes began about 10 years ago, e.g. [5][6]. Acceptance was achieved differently in each country, and this depended upon the willingness to change, perceived needs and involvement of researchers in code-writing bodies. It also became apparent that design practice can be influenced only through codes; research findings, no matter how abundant and multi-sourced, rarely influence design decisions.

Since revisions of national codes followed different procedures, it became necessary to harmonize concepts. A first step in this direction was carried out through development of the ECCS Recommendations for the fatigue design of steel structures [7]. Subsequently, these Recommendations were used as a basis for chapter 9 of Eurocode 3 "Design of Steel Structures" [8]. Although they contain a number of compromises, these documents provide a common starting point for basic research and for further development of fatigue design rules.

Previously, developments in fatigue assessment methods followed, for the most part, a simple progression from research to standardisation to design (Figure 1a). Currently, such progression is no longer practical and may be dangerous. Design requirements as well as needs for simplification influence standardisation. However, in order to simplify without becoming too conservative, more research may be needed. During code-writing processes, the limits of knowledge become apparent which in turn guides research. True interaction (Figure 1c) is a goal; mutual interaction (Figure 1b) is occurring more and more frequently. This paper gives a few examples of how these interactions are taking place.

Section two of this paper describes some of the most important aspects of the ECCS Recommendations [7]. These aspects include interaction between research and standardisation as well as the designers need for simple rules.

Even though recommendations are now available, designers often do not consider the fatigue limit state during early stages in the design process. Section three of this paper is thus devoted to good fatigue design practice. Much of this discussion was inspired by fatigue research.

Finally, a short overview of needs for further developments in structural engineering is provided. It will be pointed out how true interaction between research, standardisation and design could become beneficial and economical for modern bridge design.

2. Important research findings

Factors which influence the fatigue life of structures may be grouped as follows:
- geometry,
- stresses,
- environment,
- material properties.
Geometrical factors determine where and how quickly fatigue cracking in a structure may occur. These factors (Figure 2) include the overall configuration of the structure, span of its elements, spacing between elements and their orientation, joint geometry, local stress concentrations, as well as small discontinuities such as scratches, surface pitting, grinding marks, and weld-induced discontinuities.

Stresses determine whether or not fatigue cracking occurs. Unfortunately, stress parameters often possess the most statistical uncertainty in a fatigue analysis. Stress range, mean stress, residual stresses, stress sequence, lack-of-fit stresses, stresses due to thermal gradients, and movement induced stresses are important factors.

Environmental factors can induce fatigue cracking and may accelerate crack propagation through corrosion and creep mechanisms near the crack tip. Environmental factors include the effects of corrosive liquids or gases, temperature, humidity, hydrogen and irradiation. Fortunately in many cases, environmental factors can be neglected if adequate protection is provided.

Material properties determine how the structure reacts to some of the factors in the other three categories. Material properties include stress-strain behaviour, grain size and shape, hardness, energy absorption, chemical composition, homogeneity, electric potential, and microstructural discontinuities.

Different combinations of these factors create a wide range of fatigue-cracking conditions, documented by numerous research findings. Fortunately, predominant factors associated with fatigue cracking may create situations where design simplifications are possible. For example, in large welded steel structures, it is very difficult to avoid built-in stresses at potential crack locations. Residual stresses due to welding, stresses caused by distortions due to fabrication, and lack-of-fit problems during erection can induce tensile stresses near crack locations approaching the yield strength of the material. In addition, the geometrical configuration of the structure may contribute to high stresses. Differential settlement of supports, thermal gradients in the structure, and the behaviour of expansion joints contribute to built-in stresses.

The combination of all these stresses means that, even under dead load only and without traffic loading, the real levels of stress are unknown in all but very simple structures. Therefore, it should be assumed that, at critical crack locations, tensile stresses near the yield strength of the steel are present. Hence, in spite of research evidence from small specimen testing, the level of the applied mean stress and stress sequence have little importance for design considerations. As a result, applied stress range becomes the only significant factor in the stress category. Many national codes already incorporate this simplification, and it became possible only through interaction between research findings, design constraints and code writing.

Other simplifications are made possible by certain geometrical conditions of structural details, illustrated in Figure 2. Local stress concentrations are caused by welded attachments. Generally, the effect of such attachments is very important; the applied stress may be magnified by more than three times. Fortunately, such magnification is extremely localized in an area that, for typical stress ranges, is only tens of grains large. Outside this area, the stress ranges are not affected to similar degrees. This means that the structure and its components behave elastically under fatigue loading.
Furthermore, small weld-induced discontinuities are unavoidable [9]. Even rigorous quality requirements have been unsuccessful in avoiding crack-like discontinuities of dimensions less than 0.1 mm deep [10]. Such discontinuities are not easily detectable, but their presence eliminates any significant period of crack nucleation and short-crack growth, often called crack initiation. This is especially true when built-in stresses are high which is generally the case, as discussed above.

The combination of these factors has two important consequences. Firstly, a fatigue process which starts from crack-like discontinuities, and which is subject to overall elastic stress ranges, leads to propagation rates which are known to be independent of steel quality and yield strength for steels employed in large welded structures [11]. Therefore, changes in material properties do not change significantly the fatigue life - provided that the steel is of standard structural quality. This yields an important simplification and justification for international harmonization.

Secondly, the presence of crack-like discontinuities and the effect of local stress concentrations reduce greatly the influence of environmental conditions. Exposure to a mildly corrosive environment, such as humidity under a bridge over a river, does not mean that the fatigue life is reduced. For the same reason, the usual corrosion which is expected of weathering steels does not reduce the fatigue life of unpainted structures made from this type of steel.

From the above discussion, it can be concluded that only two major factors affect the fatigue life of large welded steel structures: stress range and geometry. This conclusion has been verified experimentally (Figure 3). Note that such simplifications are possible only for certain types of structures, such as bridges, or under certain conditions; generalization to all cases of fatigue cracking, for example to off-shore structures, is not justified.

3. Factors affecting the design process

In welded steel structures, the severity of geometrical effects can be reduced by judicious selection of the overall structural configuration and by careful choice of constructional details. Such selection is often referred to as "good design practice".

Over the past ten years, information from research encouraged the development of internationally-based design documents, for example [7]. Such documents are the result of intense interaction between researchers, designers and code writers. They represent an important step toward simplification and international harmonization and this is clearly desirable. Section 3.1 illustrates some aspects of fatigue-resistant detailing.

However, some problems remain. At the design stage, engineers continue to place too much emphasis on analysis. Not enough consideration is given to the overall configuration of the structural system. During and after design, there is not enough communication between designer, fabricator, erector, and owner regarding quality assurance and transfer of new knowledge concerning the fatigue design of the structure. These two points will be discussed in more detail in sections 3.2 and 3.3, respectively.
3.1 Fatigue-resistant detailing

In general terms, detailing goals should be low stress concentrations and small discontinuities. This is achieved through avoiding sudden changes in stiffness, partial-penetration welds, load-carrying fillet welds, and intermittent welding. Note that even non-load-carrying attachments may carry load, because of compatibility conditions with base elements, sufficient to induce cracking. In such cases, built-in defects caused by lack of penetration are potential crack sites.

A goal of recent codes is to encourage good detailing. For the most part this is achieved by sorting details into groups according to design characteristics and placing the most fatigue-resistant details in the most visible positions in figures and tables. Such sorting helps the designer focus on details which are relevant to the design under consideration; alternate details are in the same table, e.g. [7]. Also, if the designer chooses the first appropriate detail he encounters when he uses the document, then this detail is likely to be the most fatigue resistant.

In reality, different things may happen during the design process. Firstly, details are usually chosen well before fatigue strength is even contemplated. Secondly, when the fatigue assessment is later carried out, the designer may not correctly classify the detail. Thirdly, if the detail does not pass the fatigue assessment, the designer may not be capable of taking appropriate action, such as selecting a better detail.

Although many attempts have been made to encourage good detailing at very early stages of design, decision making for fatigue assessment cannot be treated explicitly by design guidelines. Good detailing can only be achieved if the designer is aware of the research findings which influenced the provisions in design codes. Such complete interaction is rarely possible.

More interaction may be helped by expert systems. Such systems are currently in their prototype stage of development e.g. [13] [14]. Based on expert knowledge in the area of fatigue, these systems may help the designer classify correctly a proposed detail. Classification errors of constructional details should thus be reduced. Also, if requested, alternative designs could be proposed by expert systems.

In all cases, the key to fatigue-resistant detailing is a rational consideration of fatigue early in the design process. A fatigue check performed only after other design criteria have been employed may result in a costly and inadequate structure. This has been recognized and much importance has been given to this aspect in the ECCS Recommendations. However, appropriate detailing cannot be achieved when the overall structural configuration does not include fatigue considerations. The following section 3.2 points out some important factors for the overall configuration of a structure.

3.2 Fatigue-resistant structural systems

In general terms, bridge structures need to be designed such that they are not subject to large numbers of high stress ranges. For example, a railway-bridge structure should not be sensitive to wheel loads and dynamic effects. Wheel-load sensitivity can be reduced through avoiding short members, employing for example composite construction and using ballast on railway bridges.

The evolution of railway-bridge cross-sections is an excellent example of the development of fatigue-resistant structural systems, see Figures 4
to 6. Traditionally, direct contact between sleepers and stringers was used for riveted construction. Load transfer is achieved by relatively stiff connections between short stringers and short floor beams which, in turn, are connected to main girders. Therefore, such structures are subject directly to wheel loading and cannot easily absorb dynamic effects, see Figure 4.

Unfortunately, early welded structures were built using framing systems similar to those used in riveted structures. Since a welded joint is more sensitive to fatigue loading than a riveted joint, these structural systems are less fatigue resistant when welding is used instead of rivets. Such structures are often found to suffer fatigue cracking as they age.

A cross section of a structure which is more fatigue resistant than the one shown in Figure 4 is illustrated in Figure 5. Placement of ballast under the sleepers spreads the loading over a longer portion of the span and reduces the severity of dynamic loading. Furthermore, ballast reduces noise and facilitates track placement and maintenance. However, some short elements remain and therefore, bolted connections should be used because of their high fatigue strength.

These designs can be improved further by use of a ballasted concrete trough in composite action with the main girders, see Figure 6. No short members are close to where the load is introduced; this configuration helps to isolate the structure from dynamic loading. Also, the use of a prefabricated deck may ease erection in difficult topographic conditions. This example shows that the choice of the overall structural configuration interacts directly with detailing and may yield fatigue-resistant design.

Although many standards have introduced ultimate-limit-state designs, structural components in bridges subjected to fatigue loading should be analysed correctly for their service behaviour. For example, partial fixity of the connection between two members should be included in the fatigue assessment. Although such fixity, for example between a stringer and a floor beam, is ignored for the ultimate-limit-state assessment, service stresses can induce fatigue cracking at such connections. The structural model used for the analysis should also be able to cope with deformation induced stresses. For example, deformation-induced stresses through lateral bracing or diaphragms may be important.

Finally, bridges should be designed in such a way that they are tolerant of potential fatigue cracks. Certain structural systems are able to redistribute loads. In the case of local fatigue cracking this may provide an early warning of a potential overall failure. Other engineering fields have termed this "fail-safe design". Several damage scenarios should be investigated, and the remaining redundancy of the structure, after a first fatigue crack, should be evaluated. However, local failure may not be acceptable as it may lead to unacceptable service behaviour. For example, fatigue cracking of a stringer in a railway bridge may lead to derailment.

In conclusion, fatigue-resistant structural systems should be developed by designers who are in a position to incorporate design experience, interpret codes correctly and profit directly from research findings. Moreover, such designers should be a driving force for true interaction between design, standards and research (Figure 10) since their knowledge and experience enables them to identify priorities for further development in all three activities.
3.3 Fabrication and erection

It is of great importance that, at the design stage, those responsible for fabrication and erection are consulted. Such consultation identifies the most appropriate specifications for criteria such as fabrication sequence, weld process and quality assurance. Also, this contact provides an opportunity to point out areas of the structure which are most sensitive to fatigue cracking, to discuss special precautions, and to become aware of fabrication and erection problems.

For example, a special precaution would be to remind fabrication and erection personnel that holes, cut out in the field and subsequently plugged without special welding procedures, have very low fatigue strengths, see Figure 7. Such practice should be forbidden without careful control. Similarly, bolt holes should not be filled with weld metal; instead, the hole should be filled with a high-strength bolt and pretensioned, thereby restoring the fatigue strength of the plate to its strength without the hole.

Lack of communication between designer, fabricator and erector is illustrated clearly by the bridge deck shown in Figure 8. In this orthotropic deck, the butt welds have a smooth profile and were fully inspected, thereby providing a high fatigue resistance. The element was lifted into place using I-sections, welded to the deck plate. Subsequently, these lifting elements were cut off above the welds. This practice counteracted the high fatigue strength provided by the butt welds of the deck plate since the welds at the I-section introduce weld discontinuities and stress concentrations which are much greater than at the butt welds. Clearly, it is wasted effort to assure high quality deck welds if the erection elements are not removed completely. The use of lifting devices should be planned and executed in accordance with the fatigue evaluation and considering the needs of the erector.

Finally, establishing communication links at an early stage encourages the transfer of information concerning the structure during post-design stages. Such information may originate from modifications during fabrication, or from last-minute decisions regarding a change in erection of the structure. In all cases, the designer should monitor closely fabrication and erection procedures. No modifications to constructional details should be allowed without approval of all parties.

Fabrication and erection are activities which require not only interaction of technical information but also, and probably most important, interaction of persons. It has been found, that lack of communication is responsible for more than 80% of problems in service and may even lead to failure.

4. Need for further work

More information is needed in many areas. Perhaps the most important field is fatigue loading and its effects on structural behavior. Actual load values, including a rational evaluation of their statistical distribution, may reduce the conservatism inherent in many designs used today. In addition, impact values for a range of situations need to be defined. Much effort should be devoted to better definitions of fatigue loading in design codes, in particular for highway bridges and crane runway girders.

An accurate definition of long-life (greater than five million cycles) behaviour is not available. More test results are needed, such as variable-amplitude and stress-history testing where a large portion of the spectrum is below the constant-amplitude fatigue limit.
Also, the suitability and reliability of fatigue-strength improvement techniques need to be identified, along with studies of why, how and how much improvement can be anticipated. For example, it is known that hammer peening, Figure 9, may increase fatigue strength [15]. However, more examination is needed in order to define its usefulness for a range of applications. In particular, the effect of variable amplitude stresses upon the compressive residual stress field, introduced by peening, needs to be studied in detail.

Analyses such as fracture mechanics should be studied with a view to identifying applications which are appropriate for large welded steel structures. Finally, a greater knowledge of fracture behaviour at relevant plate thicknesses, temperatures and loading rates would assist damage-tolerance analyses through identifying more accurately critical crack lengths.

If advances are made in the areas of fatigue loading and fatigue strength, better reliability concepts would be justified. Rationally based reliability studies using well defined data are needed to achieve economic and consistent designs. All testing, measurements and analyses should be planned so that results are useful to reliability analyses.

Fatigue studies may benefit the area of management of existing bridges. The evaluation of bridges may involve measurements, inspection, repair, strengthening and maintenance. These are all difficult tasks requiring interaction of good design practice and accurate information regarding loads, impact, fatigue strength and reliability.

5. Final remarks

Experience has shown that a lack of knowledge of appropriate fatigue-assessment procedures results in designs which may be unsafe or overly conservative. Also, some detailing may be incompatible with the goal of achieving fatigue-resistant structures. Poor design can also lead to structures which are difficult to inspect and costly to maintain.

Such difficulties arise from the fact that progress in research, standardisation and design practice took place in a serial succession. This lead to time lags of up to twenty years between development and application. Presently, the need for fatigue-resistant structures demonstrates that interactions between the three activities is needed and would be beneficial.

Clearly, areas of research should be identified from missing information within standards, and from questions raised by new and more complex designs. On the other hand, standards and codes should not be a simple collection of all available information obtained by research, regardless of their respective importance, but they should address practical needs, such as simple design rules.

Designers should not limit their source of knowledge to codes and standards, particularly when they are engaged in difficult design projects. Contacts with research institutions are of prime importance for transfer of ideas. Thus, designers will be able to be at the center of a true interaction between design experience, codified knowledge and research findings. All this will be very demanding, but the effort should be reduced with the help of intelligent computer-aided design.

Such interaction should facilitate achievement of good overall structural configurations as well as compatible fatigue-resistant detailing. Also, fabrication and erection procedures would easily be integrated with design and subsequent structural management.
6. Acknowledgments

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7. Literature

Figure 1. Relationship between research, standardisation and design.
   a) Serial succession.
   b) Mutual interaction.
   c) Complete interaction.

Figure 2. Three levels of geometrical dimensions affecting fatigue behaviour.

1. **OVERALL STRUCTURAL CONFIGURATION**
   (10^-2 to 10^0 m)
   - Concrete deck
   - Gusset attachment
   - Lateral bracing
   - Bridge girder

2. **LOCAL STRESS CONCENTRATION**
   (10^-4 to 10^-3 m)
   - Girder web
   - Weld
   - Gusset attachment

3. **SMALL DISCONTINUITIES**
   (10^-3 to 10^-4 m)
   - Surface scratch
   - Normal millscale
   - Pitting
   - Weld-induced discontinuity
   - Gusset attachment
   - Weld
   - Girder web

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Figure 3. Fatigue strength versus number of cycles for beams [12] showing.

a) Mean stress independence.

b) Steel grade independence.
Figure 4. Traditional railway bridge.
a) Cross section.
b) Typical live load stresses due to the passage of a freight train.
Figure 5. Ballasted steel trough with integrated stringers on floor beams.

Figure 6. Ballasted concrete trough in composite action with main girders.
Figure 7. A poorly executed plug of a cut out has a very low fatigue strength.

Figure 8. An orthotropic deck containing erection elements (I-sections) which were not removed entirely.

Figure 9. Improvement methods, particularly hammer peening, may increase fatigue strength.
RECENT DEVELOPMENT IN FATIGUE DESIGN RULES IN THE U.S.A.

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Summary

This paper begins with a general survey of the provisions for fatigue design of tubular structures in the AWS Code [1]. It will then focus more closely on new rules for weld profile and fatigue size effect. Finally, although the problem appears to have been put to rest at the 1987 Delft conference on Steel in Marine Structures [2], some ongoing research in this area is discussed.

1. Introduction

In the years following the Cambridge and Paris conferences [3, 4], much of the ongoing research on fatigue of tubular joints was centered in Europe and Japan. The American effort in which the author participated was primarily to digest this research and reduce it to practice. An interim report on this effort was given in the 1984 Houdremont Lecture [5], with the final results appearing in the 1986 AWS Structural Welding Code.

The present paper will be primarily a review of the fatigue provisions of the AWS code and its background in research and experience, as outlined in the following section. Since extensive background material for the subject covered here can readily be found elsewhere [2, 3, 4], we will focus on several new developments of particular interest to this conference.


The author has consistently applied an integrated approach to fracture control problems - first as a necessary adjunct to his own design work [6, 7], then in developing the US national Codes for tubular structures [8].

A well engineered structure requires that a number of factors be in reasonable balance. Factors to be considered in relation to economics and risk in the design and steel selection for tubular connections include: (1) static strength, (2) fatigue resistance, (3) homogeneity and resistance to lamellar tearing, (4) weldability. Many of these same factors arise again in setting up QC/QA programs during construction, including such issues as weld profile control and allowable flaw size to be applied during nondestructive testing [9]. Human factors and organizational issues must also be addressed, such as personnel qualifications and lines of communication/approval.

3. Fatigue Design

AWS first published design criteria for welded tubular connections in the 1972 edition of the Structural Welding Code. These included fatigue criteria (curve X") based on the use of "hot spot" stress, measured
locally at the toe of the weld joining the tubes at their intersection, or derived from a comparable theoretical analysis, e.g. finite element shells. Alternative criteria were also given for punching shear (curve "K") and brace end nominal stress. However, hot spot stress was recognized as the preferred method, as it places may different weldment types on a common basis, and allows consideration of the relevant parameters of connection geometry, provided that the designer has access to suitable stress concentration factors.

One of the early papers on hot spot stress included the following caveat [10]: "As is usual in engineering progress, the need for design rules exceeds scientific definition of variables and their interaction, and cannot wait for the day research has supplied all the data necessary to accomplish this with precision."

Two decades later, after spending over $10,000,000 on a comprehensive research program, European regulatory bodies have re-affirmed the use of hot spot stress as a basis for fatigue design. However, some of their large scale experiments fell on the unsafe side of the original AWS curve "X", giving rise to a "size effect" in the European rules.

4. Fatigue Size/Profile Effects

Early large scale test results, both welded coupons and full scale tubular connections, caused a great deal of excitement at the Cambridge conference. By the time of the Paris Conference, continuing European research had provided massive confirmation of an obvious trend of lower fatigue strength for thicker specimens of the same geometry. In many of the test series, strict geometric similarity was maintained, including the use of flat-faced weld profiles with sharp notches at their toes. While this arguably did not represent the best of design and construction practice, it did produce compelling and unambiguous experimental results, and a challenge for the theoretical analyst.

Explaining these results has required us to look beyond the hot spot stress, and has greatly expanded our conceptual understanding of the problem. The various effects of thickness on fatigue can be grouped as follows:

(1) The technological size effect, including coarser grain structure and lower yield strength, higher residual stresses and likelihood of pop-ins, increased risk of hydrogen cracking during fabrication, plane strain vs. plane stress as well as lower notch toughness, etc. These contributions to size effect might be expected to "saturate" once the relevant parameters reach their worst case.

(2) The statistical size effect, the increased risk of having a significant initial flaw with increase in stressed volume. This effect is well recognized in the design of castings, forgings, and machine parts; however, in weldments without profile control, it seems to be over-whelmed by the severe notch which is certain to occur at the weld toe.

(3) The geometrical size effect, which can be addressed by stress analysis. Notch stress theory has been applied to the initiation phase of fatigue, and fracture mechanics to the crack growth phase. As elegant as these theories are, we must remember that
they do not address aspects (1) and (2) of the problem, and thus may not account for the total size effect.

5. Notch Stress Theory

The 1984 Houdremont lecture examined the problem from the standpoint that both size and profile are important, and their interaction ought to be quantified. The notch stress theory of Lawrence et al [11] was examined, with results as re-cast in Figure 1. This shows a family of curves, with progressively lower fatigue performance for progressively more severe notch conditions at the toe of the weld. However, each curve also shows a size effect, with notch stress theory indicating log-log slopes of -0.25 to -0.40 for geometric scale-up of a given geometry. The dashed line indicates that the size effect could be mitigated by profile rules in the AWS Code, i.e. the 1/8 inch (3mm) limitation on reinforcement on butt welds.

6. Fracture Mechanics

Although many different authors have applied fracture mechanics to the problem of tubular connections, much of what will be presented here is from the Aptech joint industry project, in which the author was closely involved. Phase one was a literature review by Hayes [12].

For geometrically similar weldments (in which both the weld size and initial flaw size at the weld toe increase with thickness, and the terminal flaw size is very large), fracture mechanics crack growth analysis indicates the fatigue strength to vary with the \((1-m/2)/m\) power of thickness, where \(m\) is the exponent in the Paris Law for \(da/dN\). For \(m\) of 2.5 to 4, the power of thickness becomes -0.1 to -0.25.

Fracture mechanics can also be used to examine the effect of varying the weld geometry. Figure 2 is based on Hayes' solutions for flat-faced fillet welds with different toe angles. The stress intensity factor is sharply elevated by the notch at the toe of the weld, with elevated levels persisting for 5%-10% of the thickness. Accelerated crack growth through this region is responsible for the low fatigue performance of welds in general, and for their size effect in particular. Each geometry (except the plain plate) has an adverse size effect, even though we have improved the situation over strict geometric similarity by not scaling up the initial flaw. Smaller angle changes at the weld toe produce higher fatigue strength than more abrupt geometries, across the range of thickness variation. Applying the rule that a 1-mm wire should not pass under an 0.5T radius disc at the weld toe would require progressive improvement of the weld toe angle as thickness increases, mitigating the geometrical size effect as shown by the dashed line. Of course these fillet welds are a long way off from tubular connections, and geometrical size effect may not be the whole story, but the general insight these examples provide is valuable nonetheless.

Phase two of the Aptech JIP developed solutions specifically for the fracture mechanics analysis of tubular connections [13]. Cyclic stress intensity factor (delta-K) is computed as a function of crack size \((a)\), the reference cyclic stress range, and the geometry correction term \(Y(a)\), which is derived from a refined 2-D mesh with specialized crack tip elements, as shown in Figure 3(a). Note that the reference stress is the geometric hot spot stress, as would come from a finite element
shell analysis, so that the Y-factor only needs to account for weld detail geometry. In AWS practice, weld details (A, B, C, D) are specified as a function of local dihedral angle. Aptech developed Y-solutions for each of these, as well as a special case which occurs at the saddle position in equal-diameter connections (beta of unity). In addition to the basic flat weld profile, they also analysed improved concave profiles and less desirable convex profiles, representing a range of weld toe angles. The parameter space covered is shown in Figure 3(b). Separate solutions for shell bending, membrane stress, and punching shear, are superimposed to match the 3-D shell solution at the hot spot location. Notch effects elevate delta-K in the first 5% of growth through the thickness, and delta-K levels off at 20% of thickness, following the observed behavior of tubular joints in large scale fatigue tests.

These solutions are archived, interpolated, and superimposed in a PC program, TJLIFE, which also integrates the crack growth from initial flaw to terminal failure, using repetitive application of the annual stress range distribution. An example of this analysis is presented later in the paper.

7. Which hot spot stress?

Among the various levels of detail in stress analysis which may be adopted as a basis for fatigue calculations, the hot spot stress has evolved as the most practical basis for design purposes. Hot spot stress places many different structural geometries on a common basis, ranging from butt welds in bridge girders, to nozzles in pressure vessels, to tubular joints in offshore platforms. In AWS practice, the reference stress (or strain) is the total range which would be measured by a strain gage placed adjacent to the toe of the weld, and oriented perpendicular to the weld so as to reflect the stress which will be amplified by weld toe discontinuities. This is used with an empirical S-N design curve based on measures hot spot stress and cycles to failure, in tests of realistic as-welded hardware. The effect of representative local/microscopic discontinuities at the weld toe are presumed to be built into the data base of realistic as-welded hardware.

Re-examination of the foregoing definition from a research viewpoint reveals that it is full of ambiguities. Prof. Radenkivic’s classification of the different stress levels is more rigorous, and we shall begin by restating his definitions [14].

Sigma-G is the geometrical hot spot stress, which should be invariant given relative diameters, thicknesses, and angles of the intersecting members. It presumes that a linear variation of shell bending stress is dominant in the critical regions of a tubular connection. It can be determined experimentally by extrapolation from measurements on two suitably disposed strain gages, as defined by the EEC working group. It can be determined analytically by isoparametric finite elements which reproduce the linear variation of stress adjacent to the weld (stress at the midplane intersection of thin shell analyses will not satisfy this definition). In design, parametric formulas derived from the foregoing methods may be used, e.g. Smedley or Efthymiou.

Sigma-L is a more localized stress, which includes effects of weld profile shape and size. Experimentally, it can be evaluated by
microscopic strip gages placed as close as possible to the toe of the weld. Analytically, it corresponds to finite element analyses which have been re-meshed to zoom in on the weld toe. Since we may be working in the vicinity of a notch or stress singularity, some care is required in order to maintain a consistent definition. Although sigma-L requires more work on the part of the stress analyst, uncontrolled local perturbations above the reference stress tend to be minimized.

It is an unfortunate source of confusion that the AWS definition of hot spot stress falls loosely somewhere between these two. However, in the foregoing discussion of weld profile and size effects in terms of fracture mechanics, sigma-G was clearly intended as the reference stress; and, whereas notch stress analysis uses sigma-L, results are presented in reference to sigma-G. When the stress analysis stops at sigma-G, weld profile and size effects must be addressed elsewhere in the design process, as these effects can be quite important. Thus we have the "size effect" adjustments to the S-N curve in the British D.O.E. rules, and the somewhat more elaborate "size and profile" provisions published in AWS D1.1-86.

In one example, from the large scale French tests [14], the strain concentration factor SNCF is 3.3 for sigma-G and 6.6 for sigma-L. In the original AWS data base, in which the tubular joints were either small scale or had welds profiled so as to achieve a smooth transition, the ambiguities in the AWS definition of hot spot stress disappear in the scatter band. Coming from this background, the author was dismayed the 2:1 escalation of local stress in the French test, and the corresponding reduction in fatigue performance.

In OTC 4866 [15], Dijkstra et al describe test results for two large scale tubular joints having the same overall geometry, with and without a specially improved weld profile which merges with the adjoining base metal. The joint with the improved profile had a three-fold longer fatigue life. This was explained in terms of extrapolated hot spot stress at the weld toe, which was reduced by 25% in the case of improved profile. However, this situational interpretation is inconsistent with the concept of sigma-G as invariant for a given connection geometry, and tends to obscure the good news that weld profiling can improve the fatigue performance of a given tubular connection.

If we must abandon the concept of invariant sigma-G, then it may be appropriate to re-examine sigma-L as a basis for dealing with size and profile effects. At the very least, it seems worthwhile to adopt a consistent standard definition of sigma-L, and to attempt an S-N correlation of recent European data on this basis.

8. Example application

In one of the author's recent design projects, following 3-D finite analysis of a large ring-stiffened tubular joint [17] the analysis continued, to examine 2-D slices of weld details at critical locations. Figure 4 shows stress results from both the 3-D linear shell analysis of the whole joint and the fine mesh analysis of a 2-D slice through the weld at the heel, detail "D". The former indicated the brace hot spot to be the critical location. However, because of the internal ring stiffener and concentrated transfer of axial load into the chord,
punching shear and shell stresses were higher inside the brace footprint, rather than outside as is usually the case.

The 2-D fine mesh analysis generally confirms the 3-D analysis at the brace hot spot, when the weld merges smoothly with the adjoining base metal and the actual weld toe location is taken into account. However, at the root of the weld, there was a nasty surprise. Because of the severe notch effect, peak stresses are three times higher than the hot spot stress calculated in the full 3-D model.

The usual design method is based on hot spot stress (sigma-G) derived from shell analysis or comparable parametric equations, with notch effects at the toe of the weld built into the S-N curve. Since the analysis of 2-D slices allows us to take an explicit look at the notch effects included in sigma-L, a different methodology must be used in translating this to fatigue life. Here we use what may be called the "SAE rule" [16], which has reportedly been used successfully over the thickness range of .375 to 8 inches (10 to 200 mm). It may be stated as follows:

1. Local stress (sigma-L) is defined as that which would be measured by an 0.25-inch strain gage straddling the notch. For geometrically similar sharp notches, the larger one will result in a higher measured stress, as more of the strain gage will be in the peak region. Thus, some size effect is included, and the SCF will vary with thickness.

2. Fatigue life is calculated with an S-N curve which is quite similar to AWS curve X-1, for 1-inch reference thickness and long lives. It should be noted that the SAE method, which uses the higher sigma-L stress, will be generally be more conservative than the AWS method, which uses the sigma-G hot spot stress with curve X.

3. The fatigue strength is corrected for a small remaining size effect, in proportion to the -0.034 power of stressed volume, or -0.1 power of thickness. The rationale is that this reflects the larger probability of initial flaws being found when larger volumes are involved.

Results of applying the SAE method to the ring-stiffened joint are shown in Figure 5, as fatigue strength (the designer's conventional sigma-G) versus thickness, for both details "A" and "D". Both details show a definite size effect, with lower fatigue strength at heavier thicknesses. Furthermore, detail "D", with its more severe notch, shows shows much lower fatigue performance across the board than detail "A".

Results of a fracture mechanics calculation of fatigue life for detail "A" are also shown in Figure 5, indicating general agreement with the SAE method. Initial flaw size of 0.01-in is consistent with magnetic particle inspection of the finished weld toe. Terminal flaw size was derived from CTOD calculations for the local hot spot stress, which at this location includes an unusually high proportion of membrane stress, in addition to shell bending and punching shear. Calculations were done with the proprietary Aptech program, TULLIFE, as described earlier. Crack growth rate used the DNV mean seawater curve with a threshold of 2.0 (ksi units). The long term stress histogram came from the design spectral fatigue analysis.
Following the Cambridge conference, AWS and API made an interim adjustment in their 1980 codes. The emphasis was on weld profile effects, with both American codes including a lower fatigue design curve for welds which do not merge smoothly with the adjoining base metal. The API curves are shown in Figure 6. This reflected the point of view that it is the severity of the notch at the toe of the weld, not thickness per se, that is responsible for the observed reduction in fatigue strength. Despite this emphasis on weld profile, quantitative guidelines were lacking.

Following leads from the Paris conference, the author examined the problem from the standpoint that both size and profile are important, and their interaction ought to be quantified. Both notch stress theory and fracture mechanics were examined, as previously discussed. Both show progressively lower fatigue performance for progressively more severe notch conditions at the toe of the weld. However, each weld geometry also shows a size effect, with notch stress theory indicating log-log slopes of -0.25 to -0.40. The dashed lines in Figures 1 and 2 indicate how the size effect could theoretically be mitigated by profile rules in the AWS Code, i.e. the 1/8 inch (3mm) limitation on reinforcement on butt welds, and the modified disc test (radius of 1/2 the branch member thickness) for tubular joints.

For 1986, the AWS D1.1 Code has defined a consistent set of weld profile requirements, which represent a natural progression with thickness for the welder to follow. These vary from flat fillet-like profiles for thin members, to concave radius-controlled profiles for thick members, as shown in Figure 7. The applicable thickness range of these profiles depends on the level of fatigue performance required by the designer, i.e. whether he has used the upper fatigue curves (solid lines in Figure 6) or the lower ones (dashed lines). In this regard, design must be integrated with QC/QA during construction, to assure that the intended weld profiles will be provided.

Confirmation tests - Rice University

As we have seen, the new AWS rules for weld profile and fatigue size effect were based largely on theoretical considerations (fracture mechanics and notch stress analysis), with only second-hand access to the proprietary European research in this area. Thus, before putting D1.1-86 into practice on a major new offshore platform, Shell commissioned a quick series of eighteen fatigue experiments, designed to explicitly follow and test the new rules. These tests were conducted in air, at Rice University [18]. API and several industry companies have jointly sponsoring a more extensive three-year series of tests at Florida Atlantic University, which is doing the same in a seawater corrosion fatigue environment, as described in the next section.

Steel for both the Rice and FAU experiments is API-2W-Grade 42 (Kawasaki TMCP type). Geometrically similar half-inch, one-inch, two-inch and four-inch thick specimens were welded in various positions, so as to get a representative variation in quality of profile. A very high level of profile workmanship was being achieved in heavy production welds by the Rice series fabricator, Bullwinkle
Constructors. This was reflected in the thickest test specimens, while the thinner specimens were intended to reflect the more lenient profile requirements of the new AWS Code, as shown in Figure 8. The 2-inch (50mm) thick specimens met the disc profile test in all welding positions. However, despite a round of practice welds, some difficulty was experienced in consistently achieving the alternative profiles permitted by AWS for thinner sections. The 1-inch (25mm) overhead welds were worse than intended, and the 0.5-inch (13mm) vertical welds were too good. Although a different Texas fabricator (Brown & Root) made the FAU specimens, similar results were achieved, with some tendency to over-weld the improved profiles.

For fatigue testing, the stem of the tee-weld was loaded in tension, so as to produce a hot spot stress range (sigma-G) of either 11 or 22-ksi (77 or 154 MPa) linear shell bending at the weld toe position. After some of the thinner specimens survived the programmed 2,000,000 cycles at 22-ksi, they were re-tested at progressively higher stresses until they failed; in plotting these results, cumulative fatigue damage versus AWS curve X-1 was used.

Interpreted test results are plotted in Figure 8 as a function of thickness, fatigue strength (sigma-G at the specified weld toe position) relative to the AWS X1 design curve. As the weld details represent AWS single-sided tubular joints for half the stem thickness, the upper thickness scale interprets the stem as two branch thicknesses back-to-back; this represents a conservative treatment of the data. The smaller, lighter symbols are runouts at 20,000,000 cycles, the heavy ones failures.

Also shown is a prediction line, plotted as sigma-G but based on SAE Rule calculations using sigma-L from Rice’s detailed finite element analysis of the three profile types. Again we see a family of curves, all showing a size effect, but with higher performance for the higher quality weld profiles, as shown by the dashed lines. The solid line reflects the AWS intent, that is to maintain a more-or-less constant level of fatigue performance over a range of thicknesses by varying the weld profile. Extending the basic profile to heavy thicknesses beyond its intended range would be expected to incur a significant size effect penalty.

The data all fall on the safe side of the AWS-X1 design curve, and even further on the safe side of the SAE Rule prediction. Relaxation of the disc test profile requirements for thinner sections appears to be fully justified. Indeed, the thinnest specimens show a greater scatter towards fatigue lives on the high side, suggesting that the size effect in air may be somewhat stronger than what the profile relaxation anticipated.

11. Results to Date - Florida Atlantic

The API-JIP program of size/profile tests at Florida Atlantic University was conceived in 1984. It has now completed two of its three planned years, and has two years yet to go, following a typical rule of research fundraising. The total program involves a large matrix of 80 tests to develop S-N data covering four thicknesses, the three types of weld profile (each made in various welding positions), and two natural sea water environments (free corrosion and minimal cathodic protection at -0.8V SCE). Test methods and results to date
have been described by Hartt [19]. Some 45 tests have been completed so far.

Specimens are loaded with the stem of the tee in tension, at a stress ratio $R$ of 0.1, with a cycling rate of 0.3 Hz. Individual tests can last several months. To save machine time, several specimens are being tested in parallel, necessitating stroke-controlled loading. Crack growth behavior observed in tubular connections appears to be intermediate between load-controlled and displacement-controlled solutions, with much of the life spent in propagating small cracks unaffected by the difference. To avoid any question of unrealistically long lives due to the mode of loading, failure was defined as cracking halfway through the thickness: this is also close to the critical crack depth at yield for the 4-in (100-mm) steel with a specified CTOD of 0.010-in (0.25-mm).

Results in S-N format are presented in Figures 9 and 10. These data are for free corrosion, which show less variability than the cathodic protection data. Hartt’s use of the specimen centerline stress as hotspot stress is consistent with the use of thin shell finite element methods in design, or parametric SCF equations based on such methods e.g. Kuang’s. (Be patient - CP data and a deBack-style interpretation will come later.)

Welds meeting AWS Level I profile requirements, comparable to the Rice series, generally fall on the safe side of the AWS-X1 (API-X) design curve, with the data having an S-N slope comparable to the design curve. The 4-in (100-mm) thickness would require a size-effect reduction of fatigue strength according to the Code, so its lower performance is no surprise. The one premature failure in the 2-in (50-mm) thickness was traced to a momentary lapse in meeting the profile quality of the rest of the welding, illustrating one of the pitfalls of counting on weld profile for improved performance.

Flat-faced welds, made according to the AWS basic detail regardless of thickness, show lower performance and a more severe size effect, also as expected. Their S-N slope is comparable to the AWS-X2 (API X-prime) curve for welds without profile control. Recall that curve X-2 is substantially lower than curve X-1 (as much as 32% in Fig. 6, up to 38% as plotted in the AWS Code).

A more direct comparison of size and profile effects, versus the AWS Code, can be found in Figures 11 and 12. The fact that the S-N data agrees with the slope of the design curves facilitates conversion to this format. Both free corrosion and cathodically protected data are now shown. The interpretation of invariant hotspot stress $\sigma_{G}$ is now at the toe of the specified weld, consistent with treatment of the Rice data and with modern definitions of hot spot stress (EEC-WG3, analysis with finite thickness isoparametric elements, or use of Efthymiou SCF equations). For the specimen geometry tested, this is 90% of the centerline stress, resulting in a more conservative treatment of the data. Although welders must often over-weld in order to pass the disc test (especially the less artistic ones), this is a self-compensating mechanism beyond the domain of design calculations.

For welds meeting AWS Level I requirements, the weld profile is varied with thickness, to keep substantially similar fatigue performance up to branch thicknesses of 1-in (25-mm). Beyond this thickness, unless the
weld surface is ground smooth, a size effect penalty is imposed, following the -0.25 power of thickness. In the plot, this is shown as a band, depending on whether the effective branch thickness is interpreted as T or T/2. Except for the one premature failure discussed earlier, the seawater data fall on the safe side of AWS atmospheric design criteria. For non-redundant structures, AWS applies an additional safety factor of three on life, corresponding to an additional 21% reduction in design fatigue strengths beyond what is shown in the figure, so we would even be on the safe side of the outlier.

For flat-faced welds made according to the AWS "basic" details, all the test data fall on the safe side of the lower design criteria which would apply -curve X-2 and earlier imposition of the size effect reduction. The thinnest specimens (as T/2) qualify for Level I performance, resulting in a step up in the design criteria. Both the data and the design criteria show the size effect persisting over the full range of thicknesses tested. It's in the Code.

Somewhat surprisingly, fatigue under cathodic protection does not show any consistent improvement over that under free corrosion. Indeed, the CP data show much more erratic variability, with it being possible to draw opposite conclusions from different subsets of the data (perhaps this is why one finds so much confusion in the literature). The typical observation is that CP improves fatigue performance up to crack initiation, but accelerates crack growth thereafter.

The FAU test data are interpreted against European design criteria in Figure 13. Here we apply the de Back interpretation of situational hot spot stress, using shell bending stress projected to the actual weld toe location, with reductions from centerline stress as shown in the Figure. Even so, we see a clear separation between the performance of flat-faced (basic) weld profiles and concave (alt. #2) profiles, which would be even greater in terms of the designer’s invariant hot spot stress sigma-0. The two trend lines are shown with different slopes, as predicted by notch stress theory; they appear to merge in moderate thicknesses where earlier investigators found little difference due to weld profile, but become more widely separated at gargantuan thicknesses.

Interpretation against the U.K. Department of Energy "T" curve [20] is also shown. Here the shaded band represents the difference between rules for free corrosion and cathodic protection. With a log-log slope of -3.0, the "T" curve is steeper than the AWS design curves, and plots higher in the range of the test data; the mismatch in slope also causes more scatter in converting S-N data to size-effect format. Much of the CP data for flat-faced basic weld profiles falls on the unsafe side of the CP design criteria. With profile control (i.e. concave Alt #2 weld details) in the heavier thicknesses, performance more consistent with the design criteria would be obtained. However, given all the criteria emphasis being on plate thickness per se, and the labor union problems which arise from making craftsmanship distinctions, there seems to be little incentive for specifying and enforcing the necessary profile control in practice.
Beyond Hot Spot Stress

When the design analysis stops at the geometric hot spot stress $\sigma_G$, both European and American Codes have found it necessary to impose plate thickness or weld size/profile corrections to their design S-N curves. The need for such arbitrary and elaborate corrections may be taken as a symptom that the underlying criteria are about to outlive their usefulness - and that it may be time to search for a more fundamental design basis, which accounts for more of the problem during analysis and requires fewer corrections. There are several possible levels of analysis beyond hot spot stress which may be contemplated, as described below.

The first is Prof. de Back's proposal to evaluate the linear trend of chord shell stress at the actual weld toe position, as he has done in his experiments. Since very few connections are actually designed by conducting model tests, we would need an analytical method for doing this. Repeating the Efthymiou parameter study [21) for a variety of non-standard weld-toe positions is conceptually possible, but practically very difficult because the thick shell finite element mesh generator only does the basic weld profile at present. An alternative method of estimating this "corrected" $\sigma_G$ using the stress gradient at the perimeter of branch members is illustrated in Figure 14. This is based on the alpha-Kellogg method [22] and the assumption that the size of the weld does not change the underlying pattern of shell moments and tractions, even though it may affect stress levels. Since this method was applied in the data reduction of Figure 13, and did not explain all the observed size/profile effects, we ought to look further.

The second level would be to use some form of $\sigma_L$, the local stress at the weld toe. The SAE rule is an example of this approach. It conservatively explained the Rice data. Strip gage measurements and finite element analyses sufficient to apply this method were also obtained as part of the FAU program, but the comparisons have not yet been completed. In design, this requires a second cycle "re-zone" analysis of the weld region, after the shell analysis of the entire connection has been done. Where the designer is content to use parametric SCF for the overall shell behavior, it might also be possible to derive and use generic $\sigma_L/\sigma_G$ factors covering a parameter space similar to that shown in Figure 3(b). Alternative definitions of $\sigma_L$, e.g. different averaging gage lengths with correspondingly different S-N curves, may also be contemplated.

The ultimate level of analysis would be to go all the way to notch stress theory for initiation, and fracture mechanics for propagation, where everything is explained (almost). Several investigators have advocated this approach, with varying degrees of rigor and success [23, 24]. The Apptech TJLIFE program is an example of its successful application, and there are others. However, it has not been a significant role in the analysis effort used to check the hundreds of nodes and thousands of hot spot locations in typical Gulf of Mexico jackets, and has only been applied to special cases.

IIW s/c XV-E, Design of Welded Tubular Structures, has recently adopted the following approach [25]:

1. weld profile effects will be incorporated in the SCF formulae;
S-N curves will include a thickness effect. This could mean either the first or second level as described above. We can hopefully look forward to the development of design criteria and compatible design tools along one or both of these lines. We can probably also look forward to continuing debate as to which one is better.

13. Summary & Conclusions

AWS fatigue design criteria, along with the recent size and profile considerations, have been described. Test data have been presented which confirm these criteria for thicknesses up to 4-in (100-mm), with a variety of weld profiles and service environments. The geometric hot spot stress, invariant sigma-G, remains as the basis of today's design practice. Alternative design methods which go beyond hot spot stress were also proposed.

Prof. de Back's contributions to these developments over the years have been enormous. Even when we seem to disagree, we each justify our existence by applying our intellect to the fearless pursuit of truth, and we end up learning much from these disagreements.

14. References

Services, Palo Alto CA, December 1984 (also see OMAE-89, the Hague)


[18] John E. Merwin, FATIGUE IN WELDED TEES, Rice Univ. rept. to Shell Oil, Houston, December 1986 (data also appears in SIMS-87 PS1)


[25] Notes of IIW s/c XV-I meeting, June 22, 1987
FIGURE 1 Size and profile effects as indicated by notch stress theory.

FIGURE 2 Size and profile effects as indicated by fracture mechanics.

FIGURE 3 (a) Refined mesh at 2-0 slice used to develop YS1 solutions

(b) PARAMETRIC RANGE OF FRACTURE MECHANICS SOLUTIONS
FIGURE 4 - Results from refined mesh finite element analysis.

FIGURE 5 - Results of applying the SAE method.

FIGURE 6 - API / AWS fatigue design curves.
FIGURE 7 - AWS weld profile requirements and corresponding branch thickness limitations.

FIGURE 8 - Interpreted results of Rice fatigue tests.
FIGURE 9 - Freely corroding fatigue data for AWS Level 1 profile specimens of all thicknesses.

FIGURE 10 - Freely corroding fatigue data for Basic specimens of all thicknesses.
FIGURE 11

FIGURE 12
FIGURE 13

FAU TESTS INTERPRETED AS PER DE BACK

FIGURE 14 - Corrected Sigma - G (one possible form) for use with AWS - X2 and 0.25 power size effect.
RECENT DEVELOPMENTS IN THE FATIGUE DESIGN RULES IN JAPAN

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Summary

The latest Japanese rules for the fatigue design of both offshore structures and building structures are summarized and discussed. In addition, recent research results on the fatigue design of cast steel nodes for offshore structures are described in the hope that a sound design basis can be established for this excellent material in the near future.

1. Introduction

Ship Classification Society of Japan (Nippon Kaiji Kyokai in Japanese, called NK hereafter) included fully revised provisions for the fatigue design of offshore structures in NK rules in 1988. These are the only public rules on this subject in Japan [1] and have once been discussed by Iida at Delft SIMS conference (See Ref. 2), which are summarized first in this paper.

Kobe Steel Ltd. has developed a total design and production system for cast steel nodes to be used in offshore structures. Their system has been successfully applied to actual structures on a commercial basis. Although this is a proprietary system, many of the research results are published by Nakamura and colleagues [3,4,5,6]. These results are useful for the future developments of fatigue design rules for cast steel nodes. Fatigue design procedures proposed by Nakamura et al. are summarized in the following section of this paper.

Finally, Architectural Institute of Japan (called AIJ hereafter) is going to publish the revised edition of the design guides for tubular structures in 1989 [7]. Although these guides are applicable only to onshore structures like large-span buildings, multi-story buildings and towers, a unique fatigue design approach proposed in these guides is briefly discussed in the last part of this paper.

2. NK Rules for Fatigue Design of Offshore Structures

NK rules recommend the fatigue design based on long-term distributions of hot-spot stress or strain. It proposes that structures should be designed to have a fatigue life of at least 20 years in principle. The procedure of fatigue analysis shown in the rules mainly follows the proposal by Iida et al.[8]. The theory that underlies this analysis is that a fatigue crack initiates at the notch root at the crack initiation life of a smooth specimen that is strain-cycled at the same strain range as the strain range sustained by the notch root. This theory was verified in many ways using plate specimens with machined notches and tubular joints with ground toes [9,10]. The same analysis was found applicable also to as-welded joints when an experimentally determined stress concentration factor is used to account for effects of microscopic notches at the weld toes [8,11]. One advantage of this analysis lies in the applicability of the same analysis to low-cycle fatigue problems in which notch roots are strain-cycled in the inelastic region. An important
disadvantage of this analysis is that fatigue lives for crack extension can be included only on the experimental basis in the estimation of fatigue lives. This disadvantage is common to any of the hot-spot stress approaches of fatigue analysis.

Long-Term Distribution of Stresses in Members

The long-term distribution of stresses in members may be determined by either of the following two methods. The first method is to utilize one of the probability distribution functions. The maximum stress range is estimated by dynamic analyses of a structure under design waves or design wave spectra. A long-term distribution of stress ranges is then determined by selecting one of the appropriate distribution functions. The following Weibull distribution is recommended to use, when no other distribution functions are found appropriate.

\[ P(\Delta S_n) = 1 - \exp[-\ln(N_t)(\Delta S_n/\Delta S_{n,\text{max}})^h] \]  

where

- \( P(\Delta S_n) \) cumulative probability of stress range
- \( \Delta S_n \) range of nominal stress on members
- \( \Delta S_{n,\text{max}} \) maximum value of \( \Delta S \) during design fatigue life
- \( N_t \) total number of stress cycles during design fatigue life
- \( h \) Weibull's shape parameter

The probability density function can be derived from the above equation using the exceedance probability as follows:

\[ Q = 1 - P(\Delta S_n) = \exp[-\ln(N_t)(\Delta S_n/\Delta S_{n,\text{max}})^h] \]  
\[ p(\Delta S_n) = -h Q \ln(Q)/\Delta S_n \]

where

- \( Q \) exceedance probability
- \( p(\Delta S_n) \) probability density of stress range

When design loads concern wave loads, \( N_t = 10^8 \) is assumed to correspond to the total number of cycles for a fatigue life of 20 years. The second method is applicable when long-term wave data is available. Spectral curves of wave heights are converted to spectral estimates of stress ranges in members by applying a transfer function for each short-term sea state. A long-term cumulative stress history is obtained as a weighted sum of all the short-term stress distributions using probabilities of occurrences given by long-term wave data.

Hot-Spot Strain Range

The hot-spot is defined as the point at geometrical discontinuity where stress becomes highest. Fatigue cracks are expected to initiate at these points. The strain range at hot-spots is calculated by using the two types of stress concentration factors \( K_{ts} \) and \( K_{tw} \). The proposed procedure for calculating the hot-spot strain range is shown below (See also Fig. 1), in which the following symbols are used:

\[ \Delta \sigma \] hot-spot stress range
\[ \Delta \varepsilon \] hot-spot strain range

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\[ \Delta S \text{ nominal hot-spot stress range} \]
\[ \Delta e \text{ nominal hot-spot strain range} \]
\[ \Delta e_n \text{ nominal strain range on members} \]
\[ K_{ts} \text{ elastic stress concentration factor due to geometry of structures} \]
\[ K_{tw} \text{ elastic stress concentration factor due to local profile change at hot-spot} \]

The stress concentration factor \( K_{ts} \) accounts for local stresses due to structural discontinuities in tubular joints and can be determined by a thin-shell analysis, strain gage measurements in a model test or by parametric equations. When a finite element analysis is used, it is recommended to assume that the hot-spot is on the cross section \( 1.5t \) away from the intersection of the chord and the brace mid-planes, where \( t \) stands for the wall thickness of the brace. Recommended mesh sizes are also shown in detail in the rule. The stress concentration factor \( K_{tw} \) accounts for weld profile effects as well as effects of microscopic notches due to non-metallic inclusions at weld toes. The value of \( K_{tw} \) may be evaluated by using a solid finite element analysis or experimental techniques.

When effects of local yielding at hot-spots is insignificant, stress and strain ranges at hot-spots can be calculated by the following equations:

\[ \Delta S = K_{ts} \Delta S_n \quad \text{and} \quad \Delta e = \Delta S / E \]  
\[ \Delta \sigma = K_{tw} \Delta S \quad \text{and} \quad \Delta \varepsilon = \Delta \sigma / E \]  

where
\[ E \text{ modulus of elasticity} \]

When the effect of local yielding is not negligible, the rules recommend a method using Stowell's formula as shown below to estimate stress and strain ranges in the inelastic region, unless other efficient techniques are applicable.

Stowell's formula can be written as

\[ K_\sigma = \frac{K_\varepsilon}{K_\varepsilon - K_t + 1} \]  

where
\[ K_t \text{ elastic stress concentration factor} \]
\[ K_\sigma \text{ stress concentration factor in plastic region} \]
\[ K_\varepsilon \text{ strain concentration factor in plastic region} \]

It is allowed to use the following equations to represent inelastic relationships between stress and strain under cyclic loading, when no such data is available on the material to be used.

\[ \varepsilon = \frac{\sigma}{E} + a((\frac{\sigma}{E})^{1/n} \]
\[ 1/n = 0.0086a_u + 0.036 \]
\[ \log(a) = -0.238 + 2.24/n \]

where
\[ \varepsilon \text{ strain} \]
\[ \sigma \text{ stress} \]
\[ a_u \text{ ultimate tensile strength of material} \]
When the hot-spot stress range is in the plastic region, the amplitudes of hot-spot stresses and strains $\Delta \sigma/2$ and $\Delta \varepsilon/2$ follow stress vs. strain relations like those given by eq. 7. Namely,

$$\frac{\Delta \varepsilon}{2} = \frac{(\Delta \sigma/2)}{E} + a(\Delta \sigma/2)^{1/n}$$

Further, hot-spot stress and strain ranges are given by

$$\Delta \sigma = K_{\sigma} \Delta S, \quad \text{and} \quad \Delta \varepsilon = K_{\varepsilon} \Delta e$$

The stress and strain concentration factors in the above equation follow Stowell's formula with $K_{t}=K_{tw}$. Thus,

$$K_{\sigma} = \frac{K_{\varepsilon}^2}{K_{\varepsilon} - K_{tw} + 1}$$

When the nominal hot-spot stress range is also in the plastic region, the stress and strain ranges $\Delta S$ and $\Delta e$ can be calculated in the same way as shown previously. The nominal hot-spot stress and strain ranges given by

$$\Delta S = K_{\sigma} \Delta S_n \quad \text{and} \quad \Delta e = K_{\varepsilon} \Delta e_n$$

can be calculated from both eqs. 6 and 7 in which $K_{t}=K_{ts}$.

A long-term distribution of hot-spot stress or strain range is derived from the long-term distribution of nominal stress.

Basic Fatigue Design Curve

The basic fatigue design curves that represents hot-spot strain range versus fatigue life relations are specified as follows:

$$N = 10^{3.711 \times \Delta \varepsilon - 3.395} \quad (\text{when } N \leq 10^7)$$

$$N = 10^{11.267 \times \Delta \varepsilon - 5.790} \quad (\text{when } N > 10^7)$$

where

$N$ fatigue life

Effects of plate thickness and corrosive environments are not included in the above equations. The fatigue strength reduction due to larger plate thickness when plate thickness is large, or due to corrosion when corrosion control schemes are insufficient, should be taken into account by designers when using these equations.

Cumulative Fatigue Damage Ratio

The cumulative fatigue damage ratio is calculated by using Palmgren-Miner's linear cumulative damage rule. The damage ratio can be given by the following equation:

$$D = \int_0^{\infty} \frac{N_{t, p}(\Delta S_n)}{N} d\Delta S$$

in which the probability density $p(\Delta S_n)$ is given by eq. 3 and $N$ represents the fatigue life as a function of the hot-spot strain range.
Effects of Weld Profile and Plate Thickness

NK rules suggest as a design standard the following two values for the stress concentration factor \( K_{tw} \) in order to account for effects of weld profile and notches at the weld toes.

\[
K_{tw} = 2.6 \quad \text{for as-welded joints}
\]
\[
K_{tw} = 1.8 \quad \text{when an appropriate post-weld treatment is made}
\]

No definite criteria are shown in NK rules as for plate thickness effect, because the drafting committee considers that existing data on size-effect is still insufficient. However, Iida is tentatively suggesting the following formulae for plate thickness effect based on Japanese test results [2], which are found to be close to recent European test results [12].

\[
\frac{\Delta S(T)}{\Delta S(22\text{mm})} = 2.21T^{-0.256} \quad \text{for specimens under axial load}
\]
\[
\frac{\Delta S(T)}{\Delta S(22\text{mm})} = 4.49T^{-0.486} \quad \text{for specimens under bending}
\]

where

\( \Delta S(T) \) fatigue strength of specimens with main plate thickness of \( T \) (The fatigue strength is given in terms of stress range at \( 2 \times 10^6 \) cycles.)

Examples of S-N Curves

Relationships between nominal hot-spot strain range and fatigue life are calculated following the recommendations in NK rules, where ultimate tensile strength of material was assumed to be 450 MPa. These curves are illustrated in Fig. 2 and compared with AWS X1 and X2 curves [13]. The fatigue curves according to NK rules are applicable over a range starting from \( 10^9 \) cycles, while those according to AWS Code are from \( 2.5 \times 10^4 \).

3. Fatigue Design of Cast Steel Nodes

Obvious advantages in cast steel nodes are found in their smooth surface and variable wall thickness at intersections between a chord and a brace or between braces. It is not difficult to keep stress concentration factors at the intersections less than 1.5. There are no notches due to welding at hot-spots. Members are welded to the nodes at positions sufficiently remote from points where stress concentrates.

Although internal defects are found more frequently in cast steel than in rolled steel, fracture mechanics analysis indicates that in most cases defects on the surface govern fatigue lives of the nodes because stress intensity factors at internal defects are usually much smaller than those at external defects. In general the fatigue life of cast steel nodes is determined by the growth of cracks from surface defects.

Surfaces of cast nodes are inspected using a magnetic particle testing method. All the detected flaws of an unacceptable size are repaired by grinding and/or by welding. The nodes are then stress-relieved in a large-sized furnace. These are ordinary manufacturing processes for cast steel nodes. Residual stresses locked in completed nodes is considered to be lower than 20% of the yield stress of cast material.
Proposed S-N Relationships Based on Fracture Mechanics

Nakamura et al. used the following equation for a crack growing from a surface defect in the through-thickness direction.

\[
\frac{da}{dN} = B(1-R_e)^{-y_m} [\Delta K^{m} - \Delta K_{th}^{m}]
\]

in which

- \( \frac{da}{dN} \) cyclic crack growth rate
- \( \Delta K \) stress intensity factor range
- \( \Delta K_{th} \) threshold value of \( \Delta K \)
- \( B, y, m \) material constants
- \( R_e \) effective stress ratio given by
  - \( R_e = R \) for \(-1 < R < R_c\)
  - \( R_e = R_c \) for \( R_c < R < 1 \)
- \( R \) stress ratio
- \( R_c \) upper bound of effective stress ratio

The effective stress ratio is a function of the residual stress and the ratio of the minimum to maximum stresses due to fluctuating loads, which can be written as

\[
R_e = \frac{\sigma_R / (1-R) + \sigma_r}{\sigma / (1-R) + \sigma_r}
\]

where

- \( \sigma_r \) residual stress
- \( \sigma_0 \) stress ratio due to fluctuating loads

The stress intensity factor range \( \Delta K \) is given as

\[
\Delta K = f \sigma^{1/2} \Delta S
\]

where

- \( \Delta S \) stress range at point where crack is assumed to exist (equivalent to the nominal hot-spot stress range \( \Delta S \) defined in the previous section)
- \( f \) correction factor to account for finite geometric dimensions of cracks and nodes

Since the threshold stress intensity factor may vanish under corrosive environments, it is assumed that \( \Delta K_{th} = 0 \) in the following analysis. Further, \( f, \sigma_0, \sigma_r \) and \( \Delta S \) are assumed to be independent of crack length. These assumptions not only simplify the analysis but also result in a conservative estimate of fatigue life. Then, it follows from eq. 15 that

\[
N = A[(1-R_e)^{-y_m} \Delta S]^{-m}
\]

where

\[
A = B^{-1} f^{-m} \int_{a_i}^{a_f} a^{-m/2} da
\]

in which

- \( a_f \) critical crack length at which node is judged to have failed
- \( a_i \) initial crack length

Since existing data on fatigue of cast steel is still insufficient to
perform statistical evaluation of material constants included in the above S-N relationships, Nakamura et al. have decided the following design values based on test results obtained in Kobe Steel and also in existing literature.

An ASTM E647 crack growth test was carried out in air to determine the crack propagation parameters B and m. An example of the test results is shown in Fig. 3, in which a fitted mean line and an upper bound of dispersion as given by two standard deviations are shown. In order to allow for possible greater crack growth rate in sea water, the design value of B was decided to be three times the upper bound value of B. The values of material constants selected for use in design are:

\[ B = 2.63 \times 10^{-16}, \quad m = 4.14, \quad \gamma = 0.7, \quad R_c = 0.5 \]

The critical crack depth \( a_c \) was assumed conservatively to be 30 mm. Since a crack depth of 30 mm is very small compared with principal dimensions of cast nodes, the predicted fatigue life based on this critical size is close to the crack initiation life. Another consideration for design is required with regards to residual stress, because stress ratio gives a significant effect on predicted fatigue lives. It was assumed that \( \sigma_r = 0.2 \), in areas where defects were repaired by welding, while \( \sigma_r = 0.10\gamma \) in the other areas. These values are sufficiently large compared with residual stresses measured by Kobe Steel. Finally, the S-N curve given by eq. 18 was assumed to reach the endurance limit \( \Delta S_e \) at \( 2 \times 10^6 \) cycles.

The two S-N curves for which the initial defect depth \( a_i \) is either 1 or 3 mm, each being designated as CSN X1 and CSN X3, are calculated using the design values for the material constants, which can be written as:

\[
\begin{align*}
\text{CSN X1:} & \quad N = 10^{15.2 \Delta S^{-4.14}}, \quad \Delta S_e = 47.4 \text{ MPa} \\
\text{CSN X3:} & \quad N = 10^{14.7 \Delta S^{-4.14}}, \quad \Delta S_e = 35.1 \text{ MPa}
\end{align*}
\]

These curves are compared with other S-N curves in Fig. 2. Note that the term \((1-R_e)^{-\gamma}\) is omitted in the above equations. If \( R_e \) is not equal to zero, multiply this term to \( \Delta S \) and \( \Delta S_e \) in the above equations.

**Damage Accumulation under Long-Term Spectral Loading**

Since wave loads are random, the stress range \( \Delta S \) is the random variable. Because of simplifying assumptions included in eq. 18, a cumulative damage due to random stresses can be calculated using eq. 18 independently of the sequence of load occurrences.

Nakamura et al. utilized Palmgren-Miner's damage rule given by eq. 14 and Weibull distribution given by eq. 1 for stress range \( \Delta S \). Then, the cumulative damage ratio can be calculated as follows.

The stress range is expressed as a function of the exceedance probability:

\[
\Delta S = \Delta S_{\text{max}} \left[ -\ln(Q)/\ln(N_t) \right]^{1/h}
\]

where \( \Delta S_{\text{max}} \) life time maximum value of \( \Delta S \)

The fatigue life given by eq. 18 is thus a function of the exceedance probability. The fatigue damage ratio sustained over a spectrum of long-term stress ranges is given by eq. 14, which can be rewritten as:
Design Criteria and Fitness-for-Purpose Reject Criteria

The design and reject criteria proposed by Nakamura et al. are based partly on trade agreements rather than on a rigorous study of safety. Especially, API-X fatigue curve as shown below [14] is used as a standard for the evaluation of fatigue strength.

\[
N = 10^{15.06 \Delta S - 4.38}, \quad \Delta S_e = 34.9 \text{ MPa}
\]

However, procedures of safety assessment proposed are general and applicable to other cast steel nodes. The total number of cycles \(N_t\) in eqs. 18 and 19 was assumed to be \(2 \times 10^8\), which corresponds to 40 operating years for structures according to NK rules. The design of nodes was carried out with a target damage ratio of 1/3. An example of actual cast nodes is illustrated in Fig. 4 with finite element meshes. After nodes have been cast, defects are inspected. All the defects that do not fulfill the design target are repaired. The initial surface defect depth \(a_i\) that leads to the damage ratio of 1/3 can be calculated by repeating a numerical integration of eq. 20. The initial defect depths thus obtained are plotted against the maximum stress range \(\Delta S_{\text{max}}\) in Fig. 5 (a), in which the following values are assumed:

\[h = 1.0, \quad R_0 = -0.5, \quad \sigma_y = 0.2\sigma_y\]

In the same figure the \(a_i\) versus \(\Delta S_{\text{max}}\) curves for damage ratios of 0.1 and 1.0 are also shown for reference. The crack depth given by the D=1/3 line is called the allowable initial defect size in the report [6] by Nakamura et al. Surface defects whose depths are equal to or smaller than the allowable initial defect size may be disregarded.

The allowable defect size varies with the maximum stress range \(\Delta S_{\text{max}}\). The maximum stress range varies with the nominal stress and the stress concentration factor as shown by eq. 4. Nakamura et al. subdivided the surface of cast nodes into the three areas H, M and L depending upon the severity of stress range under fatigue loads. Area H, M or L stands for the area where stress is high, medium or low, respectively, in terms of the damage ratio calculated using API-X curve. The three areas are defined as follows:

Area H:
\[1/3 \geq D_{\text{API}} > 0.2(1/3)\]
Area M:
\[0.2(1/3) \geq D_{\text{API}} > 0.02(1/3)\]
Area L:
\[0.02(1/3) \geq D_{\text{API}}\]

where \(D_{\text{API}}\) is the fatigue damage ratio calculated using API-X curve.

The relationships between \(D_{\text{API}}\) and the maximum stress range are shown by a curve in Fig. 5 (b). Areas H, M and L are also indicated in the same figure. From this figure it is seen that the allowable initial defect sizes are: 0.9 mm in Area H; 2.3 mm in Area M; and 7.2 mm in Area L. Labor costs required for magnetic particle inspection is substantially reduced by varying the allowable defect size with the surface area. It is possible that additional defects in completed nodes are found in the fabricator's yard. These defects may have been overlooked during
manufacturing processes or produced during fabrication. Criteria is necessary to decide whether the whole node should be replaced or used with or without certain repair work. Nakamura et al. propose the "tolerable" defect sizes that are given by the D=1 line in Fig. 5 (a) as reject criteria. The tolerable defect sizes read from Fig. 5 (a) are: 2.2 mm in Area H; 5.3 mm in Area M; and 13.6 mm in Area L.

4. AIJ Fatigue Rules

All the building structures in Japan are strongly influenced by both earthquake and typhoon loads. In the structural design against these loads, fatigue damages due to cyclic loading are not taken into account, because the structures sustain large plastic deformation and loose stability when a few cycles (less than about 30 cycles) of overloads are applied. However, the effect of very low-cycle fatigue does exist and may be significant on structures especially when the structures are under the influence of dynamic loads like earthquake excitations. This subject is still unsolved and in the laboratory stage.

The fatigue provisions included in AIJ design guides for tubular structures concerned with the conventional fatigue problem pertain to structures under variable loads like crane runways. These provisions are mainly based on IIW and ECCS design recommendations [15,16]. However, a unique approach utilizing relationships between the load range divided by static capacity and the fatigue life to failure are also proposed.

An example of such S-N relationships is shown in Fig. 6 [17]. All the test results are for T-joints that failed owing to cracks grown in the chord wall along the weld toes under axial brace loading. The test results roughly fall on a straight line over a range from 30 to \(10^7\) cycles, reaching \(P_r/P_u=2\) at 30 cycles. It is interesting to see that, in low-cycle life range, resistances of T-joints approach those under static loads, demonstrating a greater resistance as the load type varies from pulsating compression to reversal and further to pulsating tension. The straight line representation of test results may have been obtained because, as joint details were varied to stiffen the chord wall, the resistance of joints was improved equally against both static and fatigue loads. Similar results are observed as for X-joints and gapped K-joints as long as failure of joints is governed by cracks in the chord walls. The mean fatigue strengths for these joints are:

\[
N = 10^{3.40(\frac{P_r}{P_u})-3.07} \quad \text{for X-joint}
\]
\[
N = 10^{3.62(\frac{P_r}{P_u})-3.94} \quad \text{for Gapped K-joint}
\]

where

\(P_r\) load range \(P_u\) ultimate capacity in compression (See Ref. 18 for ultimate capacity)

AIJ fatigue rules allow to use the 95 % confidence limits of the above S-N relationships for design. Each of these lines passes through the two points given in the table below.

<table>
<thead>
<tr>
<th>Type of Joint</th>
<th>(P_r/P_u) at (10^5) cycles</th>
<th>(P_r/P_u) at (2\times10^6) cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-joints</td>
<td>0.237</td>
<td>0.118</td>
</tr>
<tr>
<td>X-joints</td>
<td>0.193</td>
<td>0.073</td>
</tr>
<tr>
<td>Gapped K-joints</td>
<td>0.259</td>
<td>0.121</td>
</tr>
</tbody>
</table>

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The simple approach described here was first studied in 1973 [19]. No account is given of material, size and weld profile effects on fatigue life in these S-N relationships. These relationships therefore are applicable only to small-size joints in steel with a specified minimum ultimate tensile strength of 500 MPa or less. Nevertheless, these relationships are useful to estimate the fatigue strength in a low-cycle life range, where the other techniques become less dependable.

5. Conclusions

The fatigue life according to NK rules has its physical grounds in the fatigue crack initiation life at hot-spots. The hot-spot is defined as the points where cracks initiate. The procedure of fatigue analysis is simple and straightforward. However, estimated fatigue lives according to NK rules are not far from those given in other famous rules like API and AWS.

AIJ rules follow IIW fatigue design recommendations. However, it proposes also a simple procedure to predict fatigue strength given as a ratio to static capacity of joints.

Nakamura et al. proposed a fatigue design procedure as well as criteria for quality control for cast steel nodes based on the crack propagation life. In most cases fatigue cracks grow from surface defects.

As can be seen in Nakamura's analysis results with all the conservative assumptions used in the analysis, cast steel nodes apparently show many advantages over welded joints under fatigue loads. Further research efforts are required to make best use of cast steel.

Acknowledgments

The author wishes to thank Professor Kunihiro Iida of Shibaura Institute of Technology and Mr. Kenichi Nakamura of Kobe Steel, Ltd. for providing many documents with valuable suggestions.

References

for fatigue design procedure for offshore tubular connections, Int. Inst. of Welding, Doc. XIII-1020-81 and XV-297-81, July 1981


Figure 1. Definition of hot-spot strain ranges.

Figure 2. Various S-N curves.
\[ \Delta a / \Delta n = 2.63 \times 10^{-16} (\Delta K)^{4.14} \]

Test result.

CT specimen ASTM E647 in air, R=0, 20Hz
No. of specimens: 5

--- mean line
--- 2SD line
--- 2SD x3 line

Stress intensity factor range \( \Delta K \) (N/mm\(^{3/2} \))

Figure 3. Crack growth data and design curves.

Figure 4. Example of cast nodes and finite element meshes (elements=3552).
Figure 5. Defect evaluation chart.

(a) Maximum hot-spot stress range vs. initial defect size curves for given damage ratio.
(b) Maximum hot-spot stress range vs. damage ratio curve calculated using API X curve.
Figure 6. Fatigue test results for tubular T-joints.
(Fatigue strength is represented as a ratio to static strength.)
RECENT DEVELOPMENTS IN THE FATIGUE RULES IN THE NETHERLANDS

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1. Introduction

In March 1988 the Dutch code NEN 2063 "Arc Welding - Fatigue loaded structures - Calculation of welded joints in unalloyed and low-alloy steel up to and including Fe 510 (Fe 52)" was published [1]. This code followed the draft code already published in June 1983. The rules presented in this code give a general method for the fatigue assessment of steel structures and structural elements which are subjected to repeated fluctuations of loads.

Discussions in ECCS - Technical Committee 6 "Fatigue" have contributed to general methods of fatigue calculations which are in principle the same in the Recommendations of the ECCS, Eurocode 3 and the Dutch National code.

However, some differences can be distinguished in the Dutch code.

- The fatigue design curves have a constant amplitude, fatigue limit which corresponds to the fatigue strength at $N = 10^7$ cycles. At this fatigue strength the slope of the $\Delta \sigma - N$ curve changes from $m = -3$ to $m = -5$ and the corresponding value is used as a reference strength $\Delta \sigma_k$.
- For practical reasons tensile and shear stresses are used instead of the principal stresses. When acting on the same point the damages of both stresses have to be combined.
- When all stress cycles are smaller than $\Delta \sigma_k$ (the reference stress) no fatigue damage will occur. However, if only a limited number of load cycles exceed $\Delta \sigma_k$ the cycles under $\Delta \sigma_k$ (at a slope of $m = -5$) will not fully contribute to the damage. Therefore, in the Dutch code this damage ($D_5$) depends on the damage of the cycles above $\Delta \sigma_k$ ($D_3$).

These items and some others will be discussed in this paper.

2. Fatigue strength curves

The fatigue strength curves are shown in fig. 1. The classification of these lines is based on a reference stress $\Delta \sigma_k$ at 10^7 cycles. Therefore K 40 (class 40) means $\Delta \sigma_k$ ($\Delta \sigma$ at 10^7 cycles) = 40 N/mm^2.

This makes it possible to give a simple set of formulae to calculate the number of fatigue stress cycles $N$.

For $\Delta \sigma_1 \geq \Delta \sigma_k$

$$N_1 = \left(\frac{\Delta \sigma_k}{\Delta \sigma_1}\right)^3 \cdot 10^7$$

For $0.55 \Delta \sigma_k \leq \Delta \sigma_1 < \Delta \sigma_k$

$$N_1 = \left(\frac{\Delta \sigma_k}{\Delta \sigma_1}\right)^5 \cdot 10^7$$
For $\Delta \sigma_i < 0.55 \Delta \sigma_k$, 

$$N_i = \infty$$

No damage will occur if all the cycles $\Delta \sigma_i$ are less than $\Delta \sigma_k$ (fatigue limit for constant amplitude loading - dotted lines).

To exclude low cycle fatigue two limitations have been included:

$$\Delta \sigma_v < 1.5 \text{ Re}$$

$$\Delta \sigma_v < 10 \Delta \sigma_k \quad \text{or} \quad N_{\text{min}} = 10^4$$

in which: $\Delta \sigma_v$ : highest stress range in the design spectrum (see chapter 3)

$\text{Re}$ : yield strength of the material.

Taking $\Delta \sigma_k$ at $10^7$ cycles in stead of e.g. $5 \times 10^6$ is based on the fact that many results of tests can be found, especially at the lower class details, which show a fatigue limit at a number of cycles as high as $10^7$. This is even more true looking to results of larger components and thicker plates, which are used more often nowadays. Maybe a thorough analysis of all available variable amplitude test evidence on components and plates of different configurations, sizes and thicknesses will lead to a more refined conclusion.

3. Damage rule

In the Dutch code the fatigue assessment of a variable amplitude loading is based on the Palmgren-Miner rule of cumulated damage, which is in agreement with other codes.

$$D = \sum \frac{n_i}{N_i} \leq 1.$$ 

However, the statement, already mentioned in chapter 2, that there is no fatigue damage if all the cycles $\Delta \sigma_i$ are below $\Delta \sigma_k$ causes a discrepancy in the damage calculation. A single load cycle exceeding $\Delta \sigma_k$ will in that case cause full damage ($D5$) of all the cycles for which $0.55 \Delta \sigma_k \leq \Delta \sigma_i < \Delta \sigma_k$.

To avoid this discrepancy and consequent discussions about the value of the highest cycle (under or above $\Delta \sigma_k$?) the Dutch committee, writing the code, has decided to make the damage of cycles under $\Delta \sigma_k$ ($D3$) dependent on the damage of the cycles above $\Delta \sigma_k$ ($D5$). Therefore, the damage rule in the Dutch code has been written as:

$$D_{\text{tot}} = D_3 + D_5 = \sum \frac{n_i}{N_i} \frac{3}{3} + \phi \sum \frac{n_i}{N_i} \frac{5}{5} \leq 1$$

in which

$$\phi = \frac{25 D_3 (1 - D_3)}{15 D_3 + 1}$$

A graph and a table giving the relation between $\phi$ and $D3$ is given in fig. 2. It can be seen that for $D3 \geq 0.2 \quad \phi = 1$, so the full damage $D5$ has to be taken into account.

For a small number of cycles above $\Delta \sigma_k$ the damage $D3$ is so small that approximately $\phi = 0$ and no damage $D5$ will occur.
The Dutch committee is of the opinion that this formula streamlines the damage calculation, prevents discussions and is on the safe side. An example of a calculation is given in Fig. 3.

Fig. 4 gives a stress spectrum which is schematized to a design spectrum. That all stress ranges smaller than $\Delta \sigma_k$ can be neglected is a direct logic result of the design-lines. However, that $\Delta \sigma_1$, the highest fatigue stress range in the design spectrum, is defined as the stress range which will be exceeded 100 times in the service life of the structure may need some clarification. The reasons for doing are mainly pragmatic. The Dutch committee wanted to take away the slightest impression that the highest fatigue stress range might be taken equal to the highest "static" stress range. One should take in mind that the expected occurrence of the maximum static load is low; in most occasions far less than once in the lifetime. Therefore there is no direct relation between this "load cycle" and the maximum fatigue load cycle, which has a probability of occurrence of 1. To avoid discussions about this item the maximum fatigue load cycle $\Delta \sigma_1$ in the design spectrum has been defined as mentioned before.

Furthermore, this approach allows calculation of the fatigue damage for a fixed period (e.g. a one year spectrum) and multiply the value obtained by the number of years in the service life (e.g. 100 years) without discussions about the correctness of this calculation. The only disadvantage of this approach is that some people may take the number of cycles $N = 100$ too strict. There is no need to do so, since the highest cycles only in very few occasions result in a significant contribution to the fatigue damage of a structure.

4. Stress calculations

In the parent material the following stresses have to be distinguished (Fig. 5):

- $\sigma_x$ normal stress in the continuous plate perpendicular to the weld toe
- $\sigma_y$ normal stress parallel to the weld toe
- $\sigma_z$ normal stress in the non-continuous plate perpendicular to the weld toe
- $\tau_{xy}$, $\tau_{zy}$ shear stresses

All these stresses are nominal stress ranges if there are only stress concentrations which are characteristic for the detail class itself. If not, geometric stress ranges (see later in this chapter) shall be taken into account.

The Dutch code does not use principal stresses for various reasons. The governing factor for fatigue is the stressfield at the point where the crack initiates. It is even impossible to speak about the stress at the point of initiation. This stress will strongly depend on the magnification factor we use to look to the shape of the area around this point (in three dimensions). E.g. weld toes are no straight lines.

All the stresses we use for fatigue calculations are stresses which are obtained from calculations or measurements at certain distances from the weld toe after which we extrapolate these values to the weld toe.
For simple tensile or bending tests this can easily be done by calculation. However, for more complicated joints a real extrapolation method (Fig. 6) has to be used. An extrapolation based on principle stresses will certainly not give us the principle stresses at the weld toe.

There is no real good solution for this problem and therefore the Dutch committee has decided to use normal and shear stresses. The problem that raised at that moment was how to cope with both normal and shear stresses acting and causing fatigue damage at the same point of a structure.

A rather arbitrary formula -

\[ D_y + D_z \leq 1.1 \]

- was accepted.

This means that the damages must be added, in addition to an increase of the total damage sum of 10% to 1.1.

One of the reasons for increasing the damage sum is a pragmatic one; namely to avoid unnecessary calculations when one of the stress directions is far the most important one.

The consequence of this method is that in the classification of the various typical details reference stresses \( \Delta \sigma \) (or the class K) have to be given for two normal stress directions and the shear stress (Fig. 7).

Fig. 8 gives the notations of stresses in fillet welds. The stresses in the fillet welds are to be calculated using the stresses in the parent material \( \sigma_y, \sigma_z \) and \( \tau \) (fig. 5) which is already a common approach for static design in the Netherlands. So:

1. \( \sigma_{weld/y} = \sigma_y \) (parent material) 
2. \( \sigma_{weld/z} = \frac{\sigma_z \cdot t}{2a} \)
3. \( \tau_{weld/y} = \frac{\tau \cdot t}{2a} \)

For the detail given in fig. 9 class 30 has to be used for the parent material and class 25 for the weld material. It can easily be seen from (2) that for \( a/t > 0.6 \) the crack starts at the toe of the weld and the damage of the parent material has to be calculated and for \( a/t < 0.6 \) the weld material is governing.

5. Residual stresses and R-value

It is obvious that residual stress and R-value can influence the fatigue behaviour since these stresses may dramatically change the local stress levels. Tests [2] on so-called egg-box type welded specimens (fig. 10), in which high weld stresses could develop, showed a clear influence of a stress relief heat treatment (Fig. 11 and 12).

From Fig. 11 it can be seen that there is hardly any influence of the R-value for the as welded condition, due to the high level of residual stresses. However, for the stress relieved condition, the tests show a marked influence of the R-value. (Fig. 12)
This difference can easily be explained by means of crack opening displacement measurements. Where the crack for \( R = -2.5 \) is open for nearly the full cycle for an as welded specimen (Fig. 13), the crack is closed in the compressive part of the cycle for a stress relieved specimen (Fig. 14). Also other investigations e.g. [3] show similar effects for stress relieving under partial compressive loading.

Nevertheless there is no bonus for stress relieving in the Dutch code, even not in the case of partial compressive loading. There are two major reasons for this conservative approach:
- In most of the structures stress relieving can only be used for parts of the structure. It is quite uncertain what will be the effect of this costly operation after erection of the whole structure.
- The calculations of the stresses and \( R \)-values are normally based upon a linear elastic stress analysis, nevertheless, a number of aspects which influence the stress behaviour are not taken into account, such as the temperature (temperature differences), settlement of the foundation, erection stresses.

Furthermore, variable amplitude stresses can (and will) change the residual stress field continuously and for that reason it is hard to predict the real level of residual stresses under complicated loading conditions during the whole service life.

6. Weld toe improvement techniques

Improving the shape of the weld toe significantly by means of a reliable and reproducible process, just by welding, has not turned out to be very successful up till now. That does not mean that a good welding procedure is unimportant. Great care and a proper inspection technique can certainly avoid unexpected failures. However, there are a number of weld toe improvement techniques which really can improve the shape of the weld toe and/or which can introduce a more favourable (compressive) initial stress. Some of these methods as grinding, TIG-dressing, plasma-dressing, shot and hammer peening, show an important improvement of the fatigue behaviour at the weld toe.

Bignonnet [4] analysed a large number of results obtained within the framework of the European Offshore Research programmes (Figs. 15, 16 and 17). He concluded that all techniques produced encouraging results with fatigue strength improvements ranging from 20% to more than 100% in air. The improvement increases with the yield strength of the material, so using high strength steels with appropriate weld finishing techniques in the fatigue sensitive areas, allows to benefit from this materials. In sea water, under free corrosion conditions, the improvement rate is lower than in air, however, the fatigue strength remains better than that of the as welded joints.

In relation to the subject of weld toe improvement techniques Bignonnet already remarked that:
- So far, most tests have been done on relatively small size specimens.
- The use of improvement techniques is subjected to a correct quality control. Improvement is directly linked to the care taken when applying the treatment, therefore this work should be performed by properly trained and qualified personnel.
To this it can be added that there are some indications that the improvement is smaller in large sections. More tests have to be done to clarify this point. Furthermore the rate of improvement will depend on the R-value and, as already stated before, the real mean stress level and so the R-value is not well known in many structures. More tests have to be executed using high R-values.

Although the Dutch code committee was convinced that improvement techniques can contribute to a better fatigue behaviour of welds, they only made the remark "that improvement by special techniques is certainly possible". The reasons for this reserved drafting are the remarks mentioned above and the difficulties they met describing the techniques in detail.

7. Size (thickness) effect

During the recent years so much has been said about the size effect, e.g. [5, 6] that the author hesitates to touch this subject in this paper again. However, there is a size effect and with the increasing importance of large and heavy structures this effect cannot be neglected. Or as Berge and Webster [5] have said, "The thickness effect as invoked in present European design codes -

\[ S_r = S_r,0(t_0/t)^{0.25} \]

- imposes a rather heavy penalty on allowable fatigue stresses in thick plated structures. As shown by more recent data an even more conservative thickness correction may be justified".

An analysis by Van Delft [7] on T-joints shows also a somewhat larger reduction (fig. 18).

It has to be remarked that all these analyses use mean S-N lines for comparing the results of tests on plates with different thicknesses. If there is a correlation between thickness effect and the size of the most severe defect at the toe of the weld (a larger defect gives a lower thickness effect), there will be also a correlation between thickness effect and scatter in the fatigue results. This means that analyzing results by using mean S-N lines in stead of design lines (lower bound results) could be conservative. However, there is no test evidence available yet to prove this.

The formula used in the Dutch code for the thickness effect is the above mentioned relation, however, with a reference plate thickness of 30 mm. The in the code given classification can be used for thicknesses up to and including 30 mm.

For \( t > 30 \text{ mm} \) a reduction factor \( \sqrt{30/t} \) has to be taken into account. This reference plate thickness of \( t = 30 \text{ mm} \) is in accordance with the largest thickness present in the analyzed results for the classification of the joints.

The code may be somewhat conservative for thicknesses smaller than 30 mm. Van Delft [8] showed in his analysis (Fig. 19 and 20) that correcting the results for thicknesses above the reference thickness (here \( t = 22 \text{ mm} \) in accordance with the DEn-rules) only will result in conservative values for thinner plates (Fig. 19) and even when this limit is ignored (Fig. 194).
20) the results of the thin walled specimens lie within the upper half of the scatter band.

An extensive analysis of all available test results has to be made to avoid unnecessary penalty on thin walled structures.

8. Literature


9. Tables and Figures

Fig. 1 Fatigue strength curves

\[ D_3 < 0.2 \quad \varphi = \frac{25 D_3 (1 - D_3)}{15 D_3 + 1} \]

\[ D_3 \geq 0.2 \quad \varphi = 1 \]

Fig. 2 Damage correction factor \( \varphi \)

<table>
<thead>
<tr>
<th>( D_3 )</th>
<th>( \varphi )</th>
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<tbody>
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<td>( \geq 0.2 )</td>
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</tr>
<tr>
<td>0.15</td>
<td>0.98</td>
</tr>
<tr>
<td>0.10</td>
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<td>0.05</td>
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<td>0</td>
</tr>
<tr>
<td>i</td>
<td>Δσ₁</td>
</tr>
<tr>
<td>---</td>
<td>-----</td>
</tr>
<tr>
<td>1</td>
<td>90</td>
</tr>
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<tr>
<td>14</td>
<td></td>
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<tr>
<td>15</td>
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</tr>
</tbody>
</table>

\[ D₃ = Σ \frac{n₁,3}{N₁,3} = 0,0542 < 0,2 \]
\[ φ = \frac{25 \ D₃ (1-D₃)}{15 \ D₃ + 1} = 0,707 \]

\[ ≥ 0,2 \quad φ = 1 \]

\[ D₅ = φ Σ \frac{n₁,5}{N₁,5} = 0,707 \times 1,32 = 0,935 \]

\[ D_{tot} = D₃ + D₅ = 0,0542 + 0,935 = 0,989 ≤ 1 \]

**fig. 3** Fatigue calculation example for a class 50 detail
Fig. 4 Design spectrum

Fig. 5 Stresses in the parent material
Fig. 6 Extrapolation of stresses in an unstiffened beam to column connection

\[ \sigma_z = \sigma_{z,\text{geometric}} = k_t \cdot \sigma_{z,\text{average}} \]

\[ k_t = \frac{\sigma_{z,\text{geometric}}}{\sigma_{z,\text{average}}} \]
type 4 Stiffeners, perpendicular to the plate

![Diagram of type 4 Stiffeners](image)

<table>
<thead>
<tr>
<th>nr.</th>
<th>$\Delta d_i$</th>
<th>$\Delta d_k$</th>
<th>$\Delta k_{15}$</th>
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<td>401</td>
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<td>Fillet welds</td>
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<td>K 40</td>
<td>K 35</td>
</tr>
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Fig. 7 A part of the classification table
Fig. 8 Stresses in fillet welds

\[ \Delta \sigma_p = \frac{\Delta F}{b \cdot t} \]

K 30

\[ \Delta \sigma_{\text{weld}} = \frac{t}{2 \mu} \cdot \Delta \sigma_p \]

K 25

Fig. 9 Joints with load bearing fillet welds
Fig. 10 Egg box type specimen  a. plate  b. specimen

Fig. 11 log $\Delta \sigma$ - log $N$ lines for the as welded condition
Fig. 12 log $\Delta \sigma$ - log $N$ lines for the stress relieved condition

Specimen N118 $\Delta F=150$ kN $R=-2.5$ Not stress-relieved

Fig. 13 Crack opening displacement measurements
Specimen N 063. $\Delta F = 210 \text{kN}, R = -2.5$. Stress-relieved

Fig. 14 Crack opening displacement measurements

Fig. 15 Influence of the base metal yield strength on the improvement of fatigue strength by grinding

Fig. 16 Influence of the base metal yield strength on the improvement of fatigue strength by hammer peening
Fig. 17 Influence of the base metal yield strength on the improvement of fatigue strength by shot peening.

Fig. 18 Stress range at 1000kc (SRa) and t = T as function of the plate thickness.

Fig. 19 Tubular joint data corrected to t = 30 mm with
\[ S = C \cdot t^{-0.25} \]
(t = thickness cracked member) For t < 22 mm correction as t = 22 mm.

Fig. 20 Tubular joint data corrected to t = 30 mm with
\[ S = C \cdot t^{-0.25} \]
No limitation.
Prof. ir. J. de Back born on the 30th of September 1924 in Amsterdam, joined the Department of Civil Engineering at the Delft University of Technology in 1951. He was nominated as professor in Steelstructures in 1972.

Since 1957 he is involved in research on Steelstructures at the Stevin-Laboratories of the Department of Civil Engineering, executing a large number of investigations, especially on the static and fatigue behaviour of bolted and welded joints. He participated in a number of common European Research Programmes, such as the large Offshore and bridge-loadings programmes.

He is a member of several technical and standardization committees in The Netherlands and various International Committees of the European Coal and Steel Community (ECSC), the European Convention for Constructional Steelwork (ECCS), the International Institute of Welding (IIW) and the Comité International pour le Développement et l'Etude de la Construction Tubulaire (CIDECT). He is author of numerous reports and publications.