Chapter 19
Fatigue of Welded Joints

19.1 Introduction

Welding of metals is applied on a very wide scale, especially for building up structures by welding of steel plates and girders of different cross sections (I-beams, U-beams, angle beams). Welding provides many structural design options which cannot be simply realized with other production techniques. Major applications are found in bridges, cranes, ships, offshore structures, pressure vessels, buildings and various types of spatial frames.

Welding as a production technique is associated with various problems, which are characteristic for welding only. As a result, the subject "welding" became practically a discipline on its own as illustrated by the existence of welding institutes and organizations, standards and design codes, journals, and an extensive literature. Within the welding discipline, much attention has been paid to problems related to different welding techniques known under general names as: arc welding, gas welding, electron beam welding, laser welding, resistance spot welding, friction welding, and more recently stir friction welding. Welded joint designs and notch effects of welds are typical for welded structures. Welded joints are also known for a number of characteristic weld defects. These defects have created new issues for non-destructive inspections (NDI), which have stimulated developments of X-ray and ultrasonic equipment. Moreover, fatigue properties of welded joints can exhibit considerable scatter because of a variety of imperfections of these joints. As a consequence, fatigue of welded joints has always been a matter of concern, but good welding practice can be specified for fatigue critical structures including non-destructive inspections of all welds.

The present section does not give a full survey of fatigue of welded joints, but instructive books have been published, e.g. by Gurney [1], Madox [2], Radai [3] and Lancaster [4]. The present text is a brief and elementary account of fatigue problems of welded joints which are related to aspects of geometries of welded joints and fatigue lives. Welding techniques and metallurgy problems are not covered. It should be understood that welding is not attractive for materials with a high static strength obtained by a heat treatment. The welding process will then destroy the heat treatment. Also, welding of thin sheet material is not popular in view of geometric distortions due to the heat flux. However, spot welding of thin sheets can be an attractive production technique for the industry.

19.2 Some general aspects

Figure 19.1 shows a sketch of a welded butt joint between two plates. The joint is welded from one side only. A few terms are recalled in this figure, and some defects are indicated. Under a cyclic tension load, the root failure (lack of penetration) is a most serious one. It can occur over a considerable distance and the defect is similar to a surface crack. The undercut at the weld toe may be serious if the profile is sharp at the bottom of the undercut. If an undercut is not present, the transition of the excess weld material[26] to the base material still gives a stress concentration at the weld toe. Slag inclusions can be serious defects for fatigue crack initiation, more than porosity due the shape of these defects. Weld defects determine the weld quality. In this respect, significant differences can exist between manually made welds and those made by automated production. The quality of a manually welding is dependent on the competence of the welding operator. It requires training, practice, and skill to make good welds. Automated welding processes have been developed specifically for fast production of long weld seams and to eliminate part of the human factor. A more homogeneous weld quality is

[26] The excess weld material is also called the reinforcement which is a strange term.

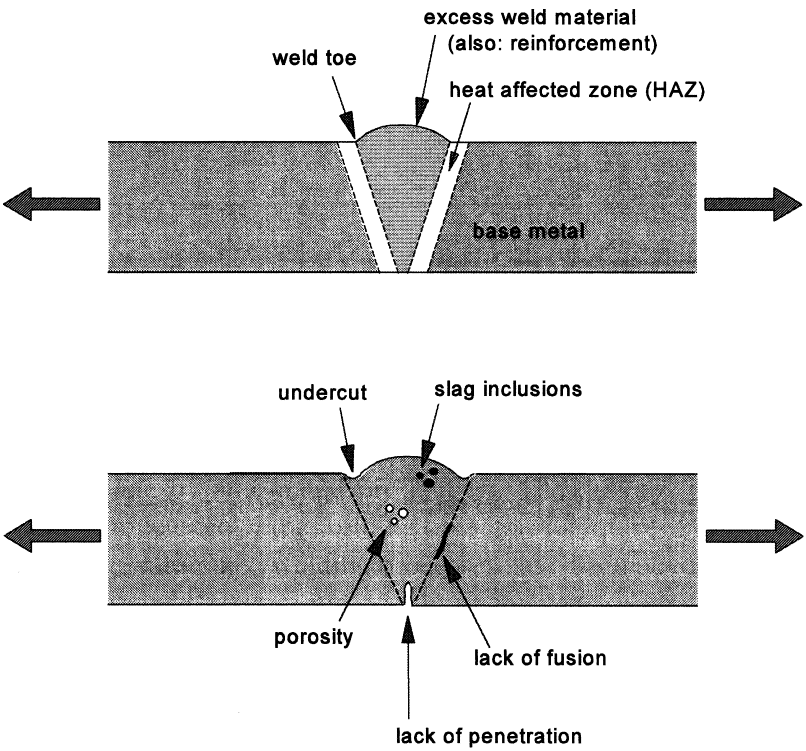


Fig. 19.1 Some terms and defects of a butt welded joint.

obtained. In any case, fatigue critical welds should always be inspected by suitable NDI techniques.

Another aspect to be mentioned here is related to thermal stresses. During welding, the weld material cools down from the melting temperature to room temperature. The weld material will contract, but this is restrained by the cooler plates. As a result, residual stresses are introduced with residual tensile stresses in the weld direction, see Figure 19.2. If such a weld is loaded in this direction, fatigue crack initiation at the ripples of the weld and related defects may be promoted. Unfortunately, residual stresses perpendicular to the weld can also be introduced by the welding process, commonly with residual tensile stress at the material surface and residual compressive stress at mid-thickness of the plate. These stresses are relevant if the fatigue load is perpendicular to the weld seam. The residual stresses introduced by welding depend on the welding technique and the design of the structure.

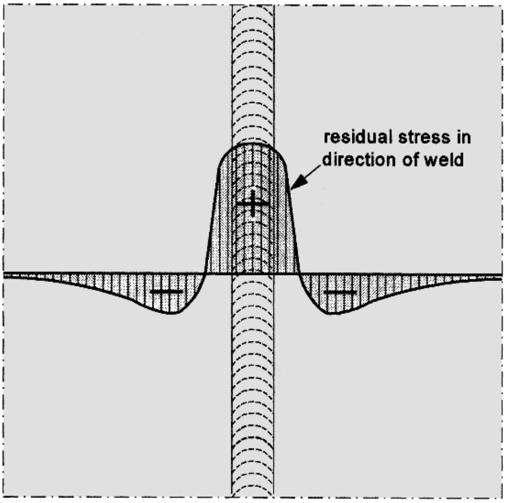
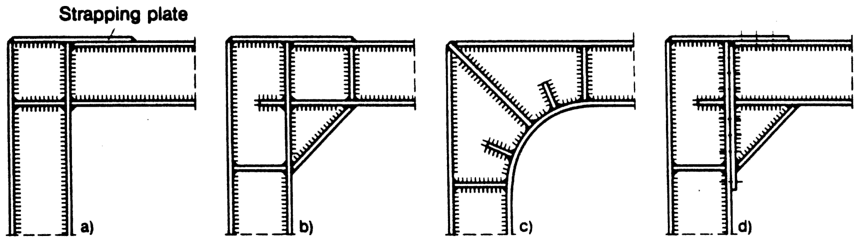


Fig. 19.2 Residual stress distribution after welding of a butt joint [2].

19.3 Geometry aspects of welds

The geometry of welded structures covers two aspects: (i) the layout of the structure, and (ii) the local geometry of the weld joint. The layout of a structure is a design problem which allows a large variety of solutions. As an illustration, Figure 19.3 shows different design options for a corner joint between two I-section girders of a frame, a plate structure of a bulge corner of the bottom of a ship, and a nozzle of a pressure vessel [3]. This figure illustrates that a variety of different solutions is available to the designer. The choice will depend on considerations like ease of production, quality to be obtained, etc. In general terms, this is a problem of production costs versus quality of the product obtained, including durability and safety. Qualitative fatigue life considerations on the corner joints between the I-section girders in Figure 19.3 suggest that solution (a) is inferior to the other three options. For the bulge corner of the bottom of a ship, the better solution should be expected to be option (b) because the location of a high stress concentration (arrow in the figure) is then separated from the location of complex weld geometries. This also applies to option (d) for the nozzle of a pressure vessel. Of course, the designer can think of more solutions than shown in Figure 19.3. It requires judgement to design against fatigue. Unfortunately, the best solution for fatigue is usually not the most profitable one for production costs.



Corner joints between two I-section girders

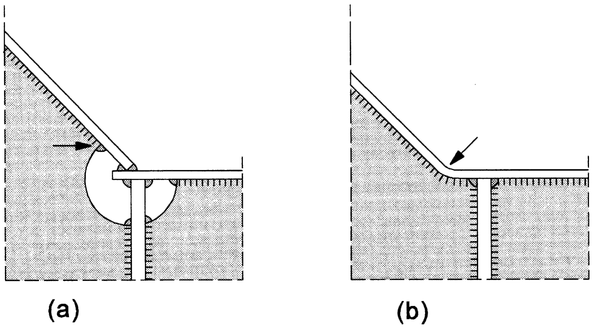
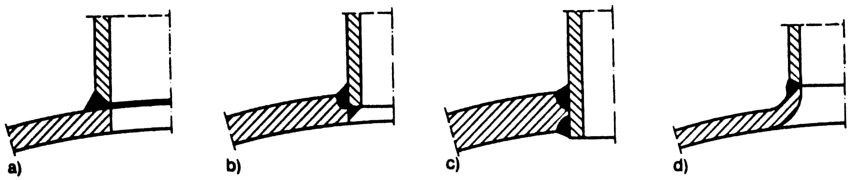


Plate structure of a bulge corner of the bottomo of a ship



Nozzle design variants of a pressure vessel

Fig. 19.3 Different designs of welded structures (figures from [3].)

The layout of the structure determines the nominal stress level of the welds. The fatigue resistance of a structure is then dependent on the geometric details of the various types of welds. The latter problem has been studied in numerous experimental research programs by fatigue tests on a variety of specimens. These specimens should simulate characteristic features of welds in structures. Most tests were carried out on steel specimens under CA loading, but later tests were also done with VA loading and other materials, notably weldable Al-alloys.

Several types of significantly different specimens are shown in Figure 19.4. Specimens (a) to (c) are simple specimens without any stress raising elements except for the weld itself. It is of some interest to consider the reduction of the fatigue strength of these specimens in comparison to the

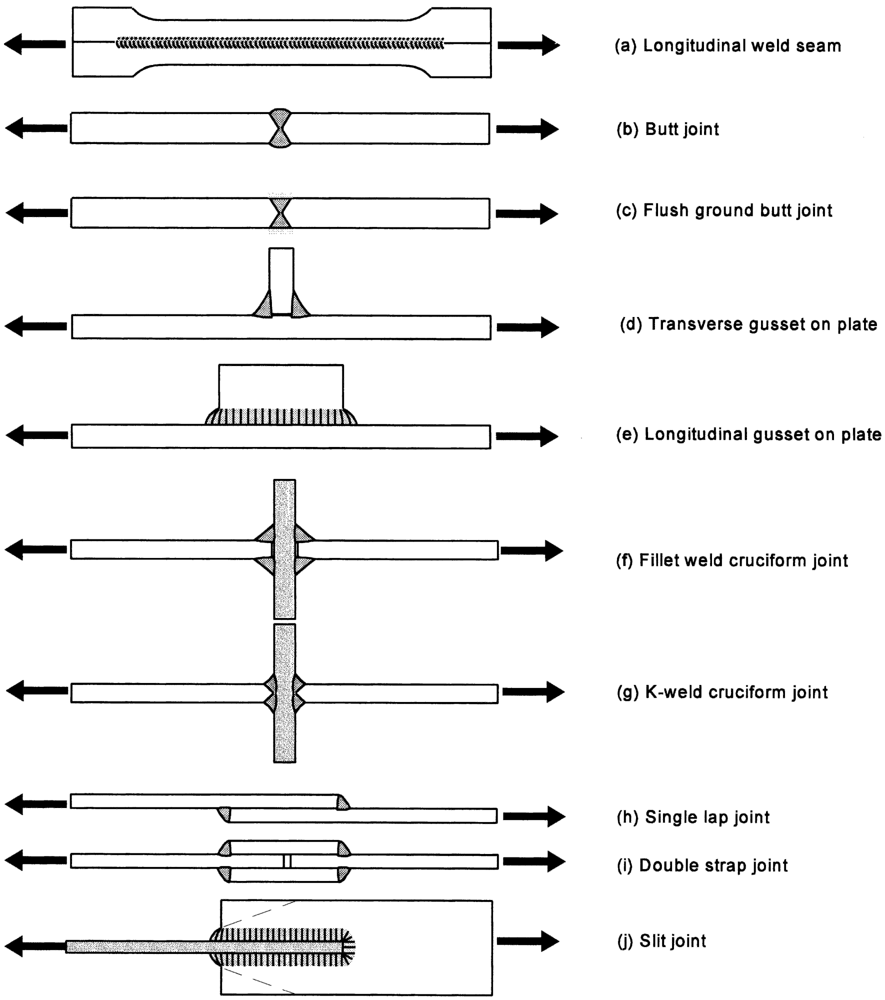


Fig. 19.4 Various specimens with typical weld configurations.

fatigue strength of the base material. The reduction of the fatigue strength is relatively small for specimen (a) with the weld seam in the loading direction. But ripples of the weld surface or weld defects may cause some reduction of the fatigue limit.

Larger reductions have been observed for specimen (b) depending on the profile of the weld reinforcement. This is illustrated by the results in Figure 19.5 which shows the fatigue strength as a function of the reinforcement angle θ defined in the graph. For $\theta = 120°$, the weld is raising fairly abruptly away form the plate surface. According to Figure 19.5, the

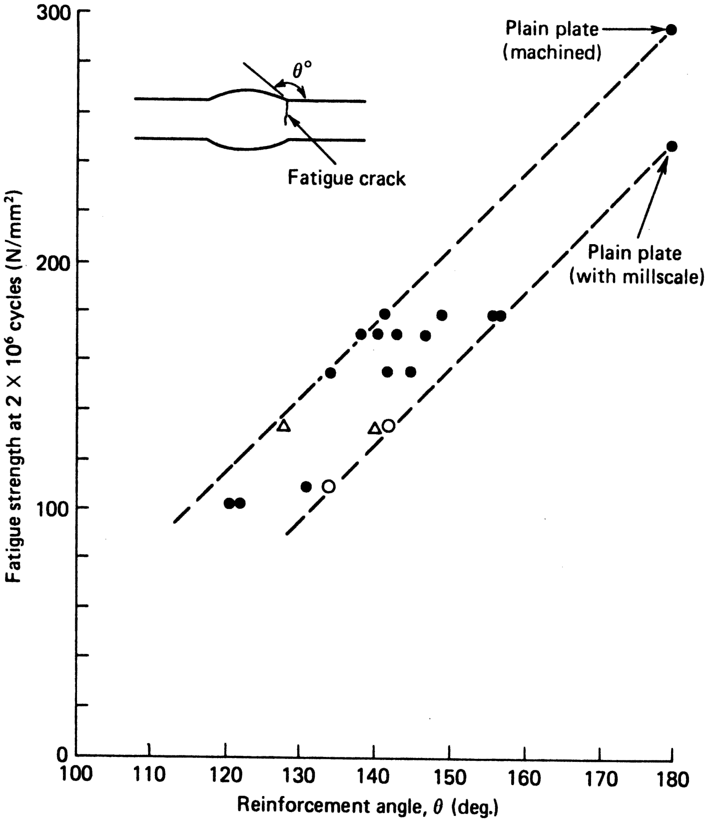


Fig. 19.5 The effect of the reinforcement angle on the fatigue strength of butt joints in steel plate [1].

fatigue strength is more than halved compared to the fatigue limit of the base material. Obviously, flush grinding of the reinforcement, specimen (c), should then improve the fatigue strength. The stress concentration of the reinforcement is eliminated. In general, a significant improvement of the fatigue strength by flush grinding is possible, but the improvement depends on possible defects in the weld material itself. Flush grinding should be more beneficial for good quality welds. In poor quality welds, crack initiation still occurs at weld defects in spite of flush grinding.

Specimens (d) and (e) of Figure 19.4 are plate specimens with a transverse and a longitudinal gusset respectively. These geometries are characteristic for several built-up structures. The fatigue strength with the transverse gusset can be moderate, although it depends on the weld toe geometry. Improvements can be obtained by removing some material at the weld

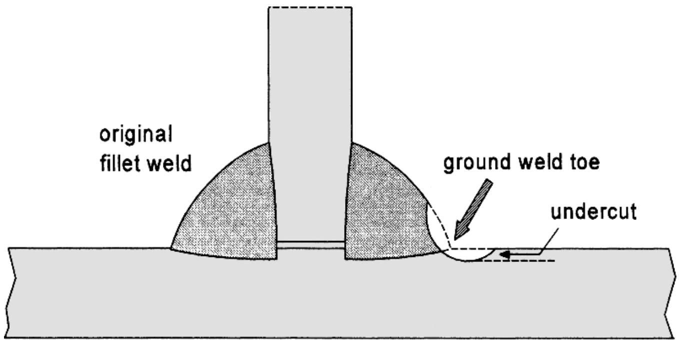


Fig. 19.6 Increased root radius of the weld toe obtained by grinding.

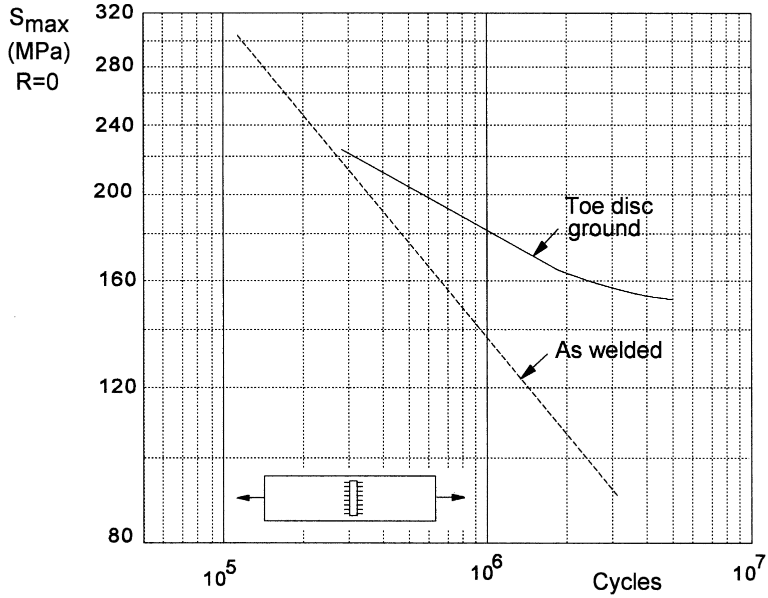


Fig. 19.7 The effect of a reduction of the toe root radius by grinding. Steel specimen with a transverse gusset [1].

toe by grinding to obtain a larger radius, see Figure 19.6. In spite of a slight undercut, the fatigue strength is increased significantly, see the results in Figure 19.7. It should be noted that the improvement is large at high endurances, whereas the improvement vanishes at low endurances. A similar trend was discussed in previous chapters (e.g. Chapter 14). The fatigue crack initiation period is relatively short at high stress amplitudes with low endurances. As a result, the major part of the fatigue life is covered by the crack growth period. Because grinding of the weld toe does not have an

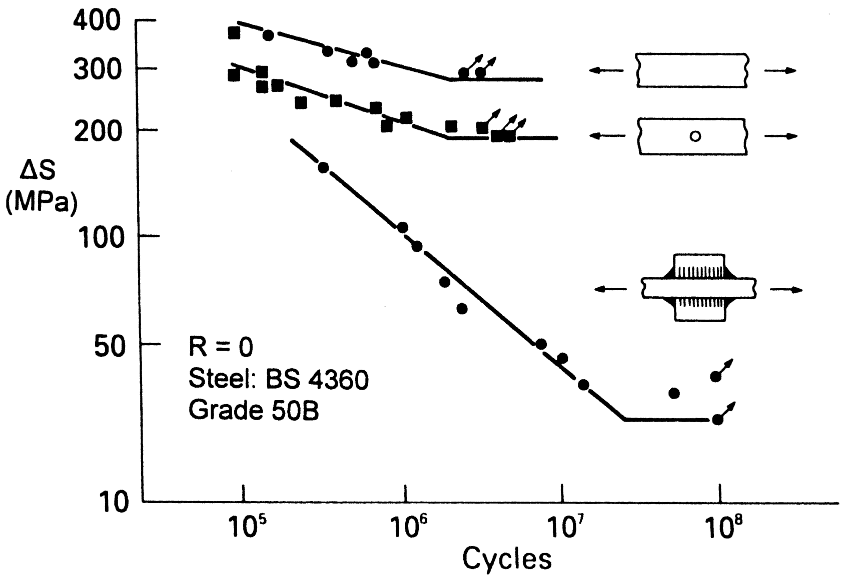


Fig. 19.8 Comparison between S-N curves of a welded specimen with a longitudinal gusset, a hole notched specimen and an unnotched specimen [2].

important influence on the crack growth period, its effect is relatively small at high stress amplitudes. But the effect can be large at low stress amplitudes with a significant crack initiation period.

An increased weld toe radius has also been obtained by so-called TIG dressing. In this process, the root of the weld is remelted to a shallow depth with a TIG welding torch (Tungsten Inert Gas), which leads to a smooth transition between the plate surface and fillet weld. According to [1], a most significant improvement of the fatigue limit is possible.

The specimen with the longitudinal gusset (specimen (e) in Figure 19.4) represents an unfavorable geometry for fatigue. The vertical gusset is causing a large discontinuity of the stiffness at the ends of the gusset. The gusset is attracting load which must be transmitted again to the plate at the ends of the gusset. A severe stress concentration can occur at this location. The effect is illustrated by Figure 19.8 by a comparison between S-N curves for an unnotched specimen, a specimen with a central hole, and a specimen with a vertical gusset. The comparison between the two upper curves indicates a moderate notch sensitivity of the base material. The fatigue limit of the hole-notched specimen is only 1.5 times lower than S_f of the base material. However, for the gusset specimen, the fatigue limit is about eight times lower, which illustrates that the fatigue limit is very low. It should also be noted that the knee in the S-N curves of the unnotched and the hole-notched specimen occurs approximately at $N = 2 \times 10^6$ cycles. However, for the gusset specimen, the knee is found at a significantly higher fatigue life, about 2×10^7 cycles. Apparently, crack initiation can occur in the gusset specimen at very low stress amplitudes, but due to slow crack growth this requires a high number of cycles before failure occurs. The same gusset specimen was also tested after a stress relieving heat treatment to eliminate residual stresses in the weld [1]. The fatigue strength at $N = 2 \times 10^7$ was raised from 37 to 55 MPa, relatively a significant improvement, but still a low fatigue limit. Residual tensile stresses in the non-stress relieved specimens have contributed to the extremely low fatigue strength at high endurances.

The two cruciform specimens, (f) and (g) in Figure 19.4, are significantly different. In specimen (f), the fillet welds leave an internal separation between the two longitudinal plates and the transverse plate. This is equivalent to having two internal cracks from which cracks can nucleate. In specimen (g), the K-weld of the two double-bevel butt ends does eliminate the internal separation. Better fatigue strength properties were reported for the latter specimen. Radai [3], citing Kaufmann, mentions a reduction of 30% of the fatigue strength if compared to the base material, whereas this percentage was 60% for the other specimen with the internal plate separation, specimen (f).

Figure 19.4 shows a single lap joint and a double strap joint to connect two plates, specimens (h) and (i) respectively. The double strap joint is free from bending and it should be expected to have a larger fatigue strength. However, a significant stress concentration still occurs at the edges of the joint. The fatigue strength is still moderate, but inferior properties should be expected for the lap joint of specimen (h).

Finally, specimen (j) in Figure 19.4 connects two plate elements which are mutually perpendicular to one another. The stress concentration is somewhat similar to the situation of specimen (e) with the longitudinal gusset. The fatigue strength should be expected to be poor. It might be improved by tapering the ends of the two plate element, see the dashed lines in Figure 19.4, which reduces the abrupt change of the stiffness.

A geometric mistake not covered by the specimens in Figure 19.4 is misalignment of welded plates. Misalignment occurs in butt joints if the central lines of the two plates are not fully parallel (small-angle misalignment), or if a small shift between these lines is present. A tension load on the joint then introduces plate bending at the weld which can significantly impair the fatigue strength. Misalignments should not occur in fatigue critical structures.

19.4 Fatigue life considerations for CA loading

The previous discussion has shown that the fatigue strength of a welded structure depends on the layout of the structure (Figure 19.3) and characteristic details of the weld illustrated by the specimens in Figure 19.4. These two aspects are interrelated because the layout may require specific types of welds. Welded structures offer their own specific design problems. Citing Maddox [2]:

> The avoidance of fatigue failure (in welded structures) is very much the province of the design engineer, including the wise choice of weld details to optimize fatigue strength, recognition of potential fatigue problems associated with welded joints and full appreciation of the fatigue loading to be experienced by the structure.

Estimations of the fatigue properties of welded structures require two inputs: (i) information on the nominal stress levels at the weld, and (ii) fatigue data of relevant welded specimens. A comparison can then be made between the structure and the welded specimen. It is assumed that similar stress levels in the structure and in the specimen will give similar fatigue lives. In essence, this is the similarity approach discussed in Chapter 7 on fatigue predictions of notched elements. This concept can be useful if the fatigue life is mainly covered by the crack initiation period. Recall that the crack initiation period includes microcrack nucleation and growth of very small cracks as long as the growth of these cracks should be considered to be a surface phenomenon (Chapter 2). The crack initiation period is then followed by the crack growth period which covers fatigue crack growth away from the material surface controlled by the crack growth resistance of the material as a bulk property independent of the material surface.

Unfortunately, fatigue cracks in a welded joint frequently start from some weld defect early in the fatigue life. For that reason, it is often thought that the crack initiation period as defined above is negligible. The fatigue life is supposed to be largely covered by crack growth only. This can be applicable if crack initiation starts at either a weld material defect or a weld geometry defect. It then is meaningful to predict the fatigue life as being a crack growth period. The crack growth life can be estimated by integration of the Paris equation: $\mathrm{d}a/\mathrm{d}N = C \cdot \Delta K^m$. The result of the integration was given previously (Eq. 8.22) which can be written as

$$\Delta S^m \cdot N = \int_{a_0}^{a_f} \frac{\mathrm{d}a}{(\beta\sqrt{\pi a})^m} = \text{constant} = C \qquad (19.1)$$

This equation is similar to the Basquin relation: $S_a^k \cdot N = \text{constant}$ (Eq. 6.2) (note that $\Delta S = 2S_a$). The equation implies a linear relation between $\log(S)$ and $\log(N)$ with the slope equal to $-1/k$. According to Eq. (19.1), k should be equal to the exponent m in the Paris equation. Analysis of large numbers of test series on welded specimens has confirmed that k is approximately equal to 3.0 [1], while values close to $m = 3$ are observed in crack growth tests on steel specimens. This result is interesting, but it should be understood that the crack growth life in Eq. (19.1) depends of the constant C. This constant is a function of a_0, the starting crack length at the beginning of the crack growth life, and a_f, the crack length at failure. As discussed in Section 8.6.1, the effect of a_f on the crack growth life is relatively small because at a large crack length the crack growth rate is high. However, the influence of the initial crack length, a_0 is large due to the low crack growth rates for a small crack length. Although the size of a_0 should be associated with the size of a weld defect, the choice of a_0 is not obvious. This question is addressed later in the discussion on a worst case analysis. Furthermore, predictions based only on fatigue crack growth can raise questions if the predicted life turns out to be very long for low stress amplitudes. In such a case, it must be expected that a substantial part of the fatigue life is spent in the crack initiation period to overcome initial thresholds for the growth of very small cracks. Unfortunately, predictions on the crack initiation period are problematic in view of the variability of the geometry of the weld toe profile, residual stresses and basic material reference fatigue data relevant for the heat affected zone (HAZ) next to the weld [5]. The definition of the crack size at the transition from the initiation period to the crack growth period is another complication.

A practical approach to the estimation of fatigue lives of welded structures is based on the development of design codes. As a result of national and international efforts, welded joint configurations have been grouped in different classes. These classes are characterized by different severities of weld configurations. As a result of analyzing large numbers of fatigue test series, an average S-N curve is adopted for each class. A survey of the codes of different countries and related literature is given by Radai in [3] and Chapter 7. The set of S-N curves given by the British Standard Institution [6] are discussed in [1, 2]. These curves shown in Figure 19.9 are discussed below. The curves are marked by capital letters as a reference to the class involved. In Class B the fatigue strength is high, whereas in Class G the fatigue strength is poor. Figure 19.9 illustrates that a large variation in the fatigue strength of welded joints exists depending on the geometry of the welded joint. The classification is based on the configuration of the joint and

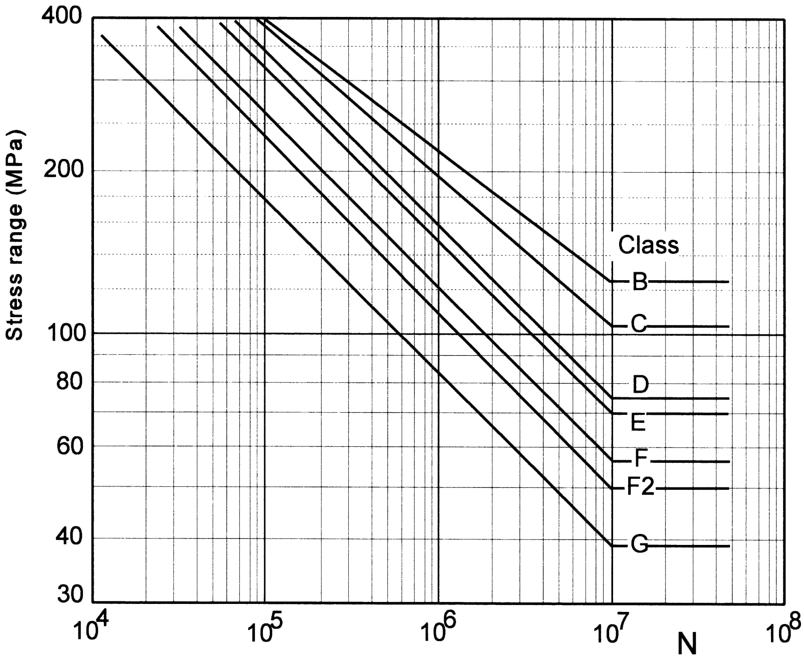


Fig. 19.9 The S-N curves for the various classes of welded joints [1, 2].

the type of the weld, provided that a good welding practice applies. A flush ground butt joint (type (c) in Figure 19.4) is suggested to be in Class B, and an as-welded butt joint (type (b) in Figure 19.4) is in Class C, the two upper curves in Figure 19.9. A transverse gusset joint (type (e) in Figure 19.4) can be in Class F, and a longitudinal gusset joint (type (f) in Figure 19.4) in Class F2, two lower curves in Figure 19.9. Joints with a rather primitive design geometry are found in class G.

Various types of joint geometries are described in the classification codes. These geometries should cover welded joint configurations as they occur in structures. The joints have a more complex geometry than shown for the specimens in Figure 19.4, which is easily recognized in Figure 19.3. As said previously, the classification does not refer to the weld quality, but to the design features of the weld geometry. However, some aspects of the weld quality are still incorporated in the classification. As an example, a butt welded joint is referred to Class C, D or E depending on the overfill profile (angle θ in Figure 19.5). Another restriction used in the classification is that a class is justified provided the weld is made by automatic welding without making intermittent stops during the welding operation. Such stops can cause weld defects on resuming the welding process. Also, NDI can be required for the high fatigue strength classes to be sure that the classification is not impaired by flaws or other weld defects. Furthermore, reference is sometimes made to the plate thickness. Experimental evidence [7] has shown that similar welded joints in thicker plates can have a lower fatigue strength. This is associated with a geometric thickness influence on stress concentrations (K_t) and stress intensity factors (K) which in the BS code [6] is called a plate thickness design penalty. As a matter of fact, it is not easy to allocate a specific class to a welded joint configuration which does not occur in the listed examples. It requires experience and engineering judgement to decide whether the S-N curves give a reasonable estimate of the fatigue strength. In addition to this conclusion, some more comments should be made on the S-N curves.

First, the S-N curves in Figure 19.9 are supposed to be valid for various structural steels with an ultimate strength in the range of 400 up to 690 MPa. Apparently, a higher static strength does not mean that the fatigue strength is increased also. This observation should probably be associated with a larger notch sensitivity and hardly any improvement of the crack growth resistance if S_U and $S_{0.2}$ are increased by modifications of the chemical composition or heat treatment.

Second, the S-N curves are based on fatigue tests carried out under a fatigue load with $S_{\min} = 0$ ($R = 0$). It is generally thought that the mean stress effect on the fatigue strength of welded structures is relatively small. This should imply that residual stresses in welded joints must also have a minor effect. However, it has been shown that the mean stress can still have a systematic effect, the more so for welded joints with a high fatigue strength. If a mean stress effect is present, it should be expected that residual stresses can also be significant. As discussed before, the fatigue strength for high endurances could be improved by a heat treatment which relieves tensile residual stresses in the welded joint. Unfortunately, residual stress distributions cannot easily be forecast for welded structures of various complexities. As a consequence, general recommendations for stress relieving treatments are questionable. However, a favorable effect of residual compressive stress was confirmed by improvements of the fatigue strength after shot peening of the toe of fillet welds.

Third, the S-N curves in Figure 19.9 are average curves, representing the mean value of scatter bands. In the BS joint classification code [6], standard deviations are suggested for each class, varying from $\sigma_{\log N} = 0.18$ to 0.25. Furthermore, it is proposed to decrease the N-values of the curves with two standard deviations, which implies fatigue life reduction factors of 2.3 and 3.2 respectively. These reductions should account for scatter of the fatigue

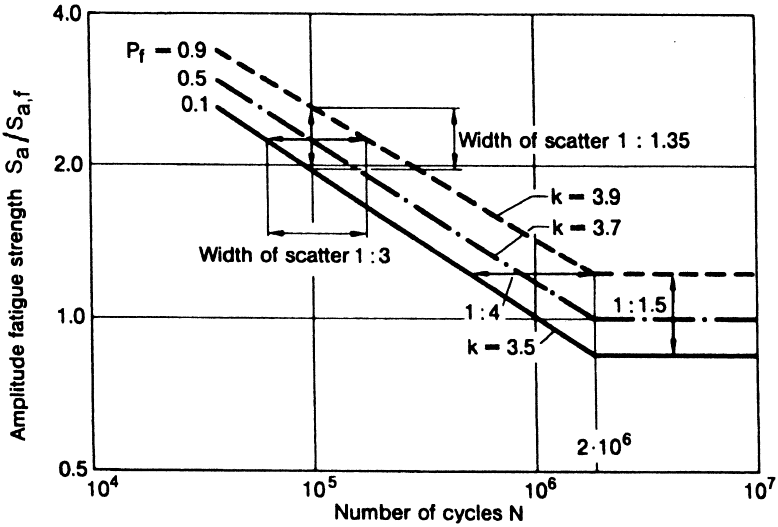


Fig. 19.10 Normalized S-N curves for welds in structural steels according to Haibach [8].

life, i.e. scatter which is considered to be representative and acceptable for variations of a normal weld quality.

An interesting graph of Haibach [8] is reproduced from [3] in Figure 19.10. Haibach proposed a normalized S-N curve obtained by dividing the stress amplitude (S_a) by the fatigue limit ($S_{a,f}$), actually the fatigue strength at $N = 2 \times 10^6$ cycles. This normalizing procedure has led to an $S_a/S_{a,f} - N$ curve, which should be valid for carbon steels with 0.12% to 0.20%C, and for different types of welded joints and R ratios. Haibach proposed scatter bands around the average curve (probability of failure $P_f = 0.50$) for probabilities of failure of 10% and 90% ($P_f = 0.10$ and 0.90 respectively). The Basquin relation was adopted, $S_a^k \cdot N =$ constant with a k-value of 3.7 for the average curve ($P_f = 0.50$). The width of the scatter band is slightly narrower at lower endurances which agrees with general experience. As a consequence, the slope factor for $P_f = 10\%$ is lower; $k = 3.5$. This value may be compared to $k = 4.0$ for Class B, $k = 3.5$ for Class C, and $k = 3.0$ for the other classes of the joint classification in Figure 19.9. The width of the scatter band, defined by N for $P_f = 0.10$ and N for $P_f = 0.90$, is given in the Haibach graph as a life ratio. The ratio is $1 : 3$ at $N = 10^5$ and $1 : 4$ at $N = 10^6$. The width between the two P_f-values corresponds to 2.58 standard deviations (normal distribution assumed). It implies that $\sigma_{\log N} = 0.186$ and 0.235 for the two ratios respectively. These values are again close to the standard deviations adopted in the joint classification code cited previously, which are in the range from 0.18 to 0.25. Apparently, a good deal of agreement is found between Haibach's analysis and the S-N curves of the BS-joint classification code.

It is also of some interest to consider scatter of the fatigue strength. The scatter band of the fatigue strength at 2×10^6 cycles in Figure 19.10 is accounted for by a ratio $1 : 1.5$. This corresponds to a standard deviation $\sigma_{\log(S_a)}$ of $0.069 \approx 7\%$.

Some further comments on fatigue life estimates of welded joints should still be made. According to the previous discussion, a fatigue life estimate for a welded joint starts with considering the joint classification. After such a choice has been made, the S-N curve follows e.g. from Figure 19.9. This curve should be reduced with two standard deviations to account for scatter. A fatigue life of the welded structures is then obtained by reading the N-value at the nominal stress level of the welded structure. Two problems are easily recognized; first the selection of the joint classification, and second, the assessment of the nominal stress level of the structure. Furthermore, corrections could be considered for the effects of mean stress and plate thickness. The result of all these steps is affected by uncertainties. With some judgement about these issues and conservative assumptions, it is possible that the estimated fatigue property is satisfactory in comparison to the design goal. This could imply that an acceptable margin of safety is still left. However, if the result does not give sufficient confidence in comparison to the design goal, exploratory fatigue tests should be considered. It then is necessary to simulate all characteristic details of the welded structure in the specimen to be tested. The fatigue life in the test provides a measure of the fatigue quality of the weld design. From this result, an S-N curve is obtained by adopting the Basquin relation using an assumed k-value, e.g. $k = 3.0$.

Of course, the best solution is to carry out a fatigue test on the structure itself with a representative service load-time history. However, FE calculations should also be considered to explore the stress distribution at critical locations in a welded structure. Sometimes strain gage measurements may be instructive. Strain gages then should be located at critical points where crack initiation may occur. However, at the root of a weld it is difficult to apply strain gages due to the irregular profile of the weld surface. Strain gages can then be applied at a small distance away from the root of a weld which has been labeled as the hot spot stress location. The indicative significance of such measurements requires a good understanding of local stress gradients.

19.5 Fatigue endurances of welded joints under VA loading

Another problem arises if the fatigue load in service is associated with VA loading. The Miner rule is generally considered to be the only calculation rule available for welded structures, although it is also claimed to be unconservative because $\sum n/N < 1$ results are found. Limitations of the Miner rule were previously discussed in Chapter 10. It was pointed out that load cycles with amplitudes below the fatigue limit can still contribute to fatigue damage. Because fatigue crack growth is an important part of finite lives of welded structures, small cycles with amplitudes below the fatigue limit can contribute to the growth of cracks initiated by cycles with amplitudes exceeding the fatigue limit. As discussed in Chapter 10, extrapolation of S-N curves below the fatigue limit must be advised for life calculations with the Miner rule. The extrapolation was shown in Figure 10.11 as line B. It implies that the Basquin relation ($S_a^k \cdot N = \text{constant}$) is assumed to be also applicable to cycles with stress amplitudes below the fatigue limit. This does not mean that the Miner prediction becomes accurate. However, it is more realistic and more conservative to account for fatigue damage contributions from small cycles.

An other proposition was made by Haibach [8], line H in Figure 10.11, with a slope factor $2k - 1$ (Basquin relation: $S_a^{2k-1} \cdot N = \text{constant}$). Later, in the welding code [6] a slope factor of $k + 2$ was proposed with the knee in the S-N curve at 10^7 cycles. Because the value of k for welded specimens is in the order of 3, the two factors are practically equal ($2k - 1 = k + 2$ for $k = 3$). Of course, a prediction with such modified S-N curves is more conservative than the original Miner rule prediction.

Gurney [9] analyzed $\sum n/N$-values obtained in a large number of VA test series on welded specimens. The $\sum n/N$-values were obtained with S-N curves extrapolated below the fatigue limit in accordance with the Basquin equation. He found as an average value $(\sum n/N)_{\text{average}} = 1.2$, while for 99% of the data $\sum n/N$ was larger than 0.35. It suggests that a rough estimate of the fatigue life can be obtained. It should still be recalled from the discussion in Chapter 10 that the Miner rule prediction does not account for any interaction effect. Miner prediction results must be considered with caution, and safety factors on life should be considered.

19.6 Two special cases

Two special cases of welded structures are related to pressure vessels and large tubular offshore structures. They are briefly addressed below.

Pressure vessel

A pressure vessel is special for the following reasons: (i) The load spectrum applicable to many pressure vessels is rather flat and can be approximated by constant-amplitude loading with a zero minimum load ($R = 0$). (ii) The required fatigue life in numbers of pressurization cycles is not excessive. The design goal may be in the order of 10^5 cycles. (iii) Safety is an important issue in view of explosive failures. Inspection of the weld quality is obligatory.

Fatigue life estimates can be made with S-N curves which are assumed to be relevant to the critical welds of the pressure vessel. These curves can give indications of the safety margins of the structure. However, in view of the limited fatigue life and safety considerations, it is desirable to know how fast a crack will grow if it is present. A fracture mechanics problem must then be considered. A "worst case" analysis should be recommended. It implies that an initial defect has to be assumed with a size depending on the limitations of NDI techniques used. The size must be related to the minimum size that will not escape detection by NDI. Crack growth should then be predicted by using relevant K solutions and crack growth data of the material involved. The crack growth life is continued until a critical K-value is reached causing failure ($K_{\max} = K_c$), or until a "leak before break" situation arises. A prediction for a semi-elliptical surface crack was discussed in Section 8.6.3, where it was assumed that a full through crack was immediately present after the crack penetrated through the full plate thickness, see Figure 8.24. In general, the geometry of the structure and an initial flaw will be more complex. As a result, it may be necessary to make more elaborate FE calculations to obtain relevant K-values. Another problem involved is associated with the leak-before-break condition. Usually, pressure vessels are made from ductile steel. At the moment of break through of the crack, considerable plastic deformation may occur. Elasto-plastic fracture mechanics should then be applied, which is not really a simple procedure. Furthermore, if the pressure vessel is filled with a fluid, the pressure will rapidly decrease after some leakage. The crack driving force decreases simultaneously and a catastrophic failure need not occur. However, this is

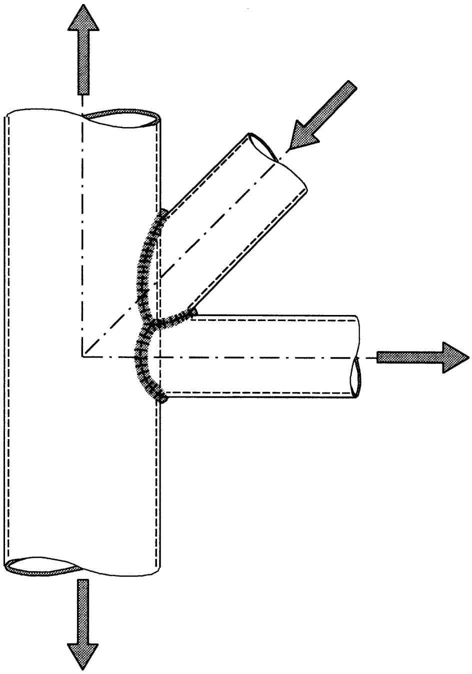


Fig. 19.11 Example of tubular welded joints.

not necessarily true for a gas filled pressure vessel. For thick-walled pressure vessels, leak-before-break is obviously preferable to a complete failure in view of safety issues, but if the thickness is large it may be difficult to satisfy the leak-before-break criterion. Of course, crack detection before a break through occurs is the better solution. Such a situation might be monitored by combining crack growth predictions, NDI and safety factors for quantitatively unknown influences.

Tubular offshore structures

An entirely different problem setting is applicable to fatigue cracks in welded tubular frames used in offshore structures. Complex nodes are present where several tubes meet. An example is schematically shown in Figure 19.11 with two side braces connected to a main pillar. The size of these tubes is immense. Diameters in the order of 2 m (6 ft) and larger are used, and wall thicknesses can be as high as 5 cm (2 inch). As a result, weld seams are long and do not occur along straight lines. Furthermore, the welded structure operates in salt water which is an aggressive environment, while the load spectrum depends on the sea waves and weather conditions. The load spectrum contains many small cycles, but in stormy weather cycles with large amplitudes occur. The fatigue life design goal may be 40 years or more, which implies a large number of cycles. Fatigue cracks are highly undesirable in view of difficult inspections and repairs. As part of the design analysis, much effort is spent on fatigue problems. Extensive FE calculations are made to obtain detailed pictures of the stress distribution in the joints. The results should indicate where the most fatigue critical areas of the welds are located. Local stress levels at these critical areas can be used for preliminary fatigue life estimates, but the similarity between the tubular nodes and simple specimens is a problematic issue. Moreover, in view of the VA load spectrum, a Miner rule calculation must be made, which introduces other uncertainties.

In some laboratories, full-scale tests are carried out on very large specimens representing a typical node joint. These tests are also carried out to study crack growth along the welds of the joints. Obviously, a scenario is also required for the very expensive offshore structures. In comparison to the pressure vessel case, the problem is more complex because of the geometries involved, the occurrence of variable-amplitude loading, and the salt water environment. Furthermore, in a welded joint of a large structure, several fatigue crack nuclei are initiated simultaneously along the weld. Initially, these cracks grow independently until they coalesce and then grow as a single crack with a large length along the weld toe. Predictions on this type of crack growth with fracture mechanics require considerable efforts.

The influence of salt water is confusing. It is well known that the fatigue limit of unnotched specimens in a salty environment is very low and almost non-existent, see Figures 2.29 and 16.1. The effect of salt water on fatigue crack growth is generally detrimental, but less disastrous than the effect on the fatigue limit. As discussed in Chapter 16, a corrosive environment can enable fatigue crack initiation at very low stress amplitudes by a surface corrosion process. Probably, this aspect is less important for fatigue of welded joints in a worst case analysis because a small crack is supposed to be present at the beginning of the fatigue life. During fatigue crack growth, the corrosion effect on the growth rate depends on the accessibility of the environment to the crack tip and the corrosion products left inside the crack. It is difficult to analyze this phenomenon in a realistic model. Comparative fatigue tests on welded steel specimens in air and in salt water under variable-amplitude loading have indicated lower endurances in salt water. In the high-cycle region, the effect was found to be a life reduction by a factor in the order of three to four. It should be understood that the magnitude of this factor is based on empirical evidence from laboratory experiments, and not from experiments in the open sea. Actually, safety factors to account for the corrosive influence cannot be chosen on rational arguments only. Engineering judgement based on understanding of possible influences, experience and economic and safety consequences of fatigue failures should lead to reasonable decisions, also with respect to corrosion protection and inspections.

19.7 Spot welded joints

Spot welded joints are entirely different from welded joints with continuous weld seams. Spot welding is a local attachment between sheets, plates or sections. As a design option, spot welded joints are more comparable to riveted or bolted joints. Spot welding is usually restricted to structural configurations with low material thicknesses which may be of the order of 1 to 2 mm (0.04 to 0.08 inches). However, spot welding can also be applied to thicker material, and fatigue evaluations of such joints between steel plates were carried out by Overbeeke and Draisma [10, 11]. Spot welding is frequently used in the automotive industry for assembling preformed sheet metal parts. The attractive feature is that spot welding allows high production rates by full automation of the production process. In the past, spot welding was also used in aircraft structures, particularly for attaching stiffeners to sheet metal skins, both made from Al-alloys. Load transfer in such joints can be negligible and the joint should not be fatigue critical. However, such joints should be carefully sealed to prevent moisture penetration and corrosion in the joint which can activate fatigue crack nucleation.

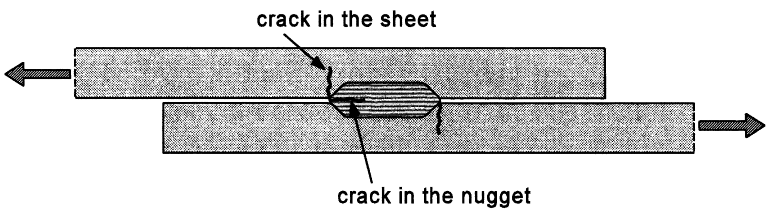


Fig. 19.12 Cracks in a spot welded joint.s

The fatigue strength of a spot welded lap joint is very low. The geometry of a spot welded joint is somewhat similar to the geometry of a riveted joint. However, contrary to a riveted joint, holes and rivets are absent in a spot welded joint. The weld nugget and the sheets are an integral part. As a consequence, the stress concentration at the edge of the nugget is high. Cracks are nucleated at this critical location, see Figure 19.12. The predominant failure mode is cracking in the sheets, but cracks in the nugget have been observed. From a fatigue point of view, spot welded joints should be considered with caution and full-scale fatigue tests are recommended. An interesting method for the evaluation of the severity of spot welds in parts of a motor car has been proposed by Rupp et al. [12].

19.8 Major topics of the present chapter

1. The fatigue behavior of welded joints is entirely different from the behavior of joints with fasteners. The fatigue critical locations of welded structures occur at the welds, while the nominal stress level at these welds depends on the layout of the welded structure. Furthermore, the variety of welding processes is large and several geometric imperfections and defects in the weld itself can occur. The S-N curve of a welded structure depends very much on the design of the joints and the quality of the welding. Preliminary information on S-N curves is given in the Welding Codes.
2. Estimates of the fatigue life of welded structures loaded by a variable-amplitude load history can be obtained with a Miner calculation. But the S-N curves should then be extended to high N-values for damage contributions of fatigue cycles with amplitudes below the fatigue limit.
3. Environmental and load frequency effects for welded structures in sea water should be accounted for by safety factors while periodic inspections are desirable.
4. A worst case analysis must be considered for welded structures if serious safety or economic problems are a relevant issue if fatigue cracks can occur. The fatigue life must be assumed to be fully covered by fatigue crack growth starting from a possible initial defect. The life prediction is then replaced by a crack growth prediction. The result is significantly depending on the assumed size of the initial defect.

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