CHAPTER

17

Fatigue-resistant Design

Introduction

The word 'fatigue' originated from the Latin expression 'fatigare' which means 'to tire'. The term fatigue was coined by Braithwaite in 1854. *Fatigue* is the initiation and propagation of microscopic cracks into macro cracks by the repeated application of stress(es) (Lu & Mäkeläinen 2003). The damage and/or failure of materials under cyclic loads in engineering applications is called *fatigue damage*. Fatigue failures generally take place at a stress much lower than the ultimate strength (yield stress) of the material—at a stress which is considered safe on the basis of static failure analysis. The failure is due primarily to the repeated stress from a maximum to a minimum. Fatigue failure may occur in many different forms: *mechanical fatigue*, when the components are under only fluctuating stress or strain; *creep fatigue*, when the components are under cyclic loading at high temperature; *thermo-mechanical fatigue*, when both mechanical loading and temperature are cyclic; *corrosion fatigue*, when the components are under cyclic loading in the presence of a chemically aggressive environment (Ramachandra Murthy 1994; Ramachandra Murthy et al. 1994).

The major loads that a building is subjected to are dead, live, wind, and earthquake loads. Dead and live loads are always present. However, the design live loads may be experienced by the structure only once or twice a year. The occurrence of the design wind and seismic loads, which are cyclic in nature, will be about 1 in 50 years. The occurrence of cyclic stresses in buildings is usually caused by machinery and the induced cyclic stresses are typically low. Steel can withstand an infinite number of load reversals at low stresses. Hence, in buildings, fatigue is not considered in the design of members. Examples of structures which are prone to fatigue are bridges, gantry girders, cranes, slender tower-like (open) structures (subject to wind oscillations), offshore platforms (subject to wave load), and structures supporting large rotating equipment. For these structures, verifications in the limit state of fatigue are often more critical than in the serviceability or ultimate limit state. This implies that the fatigue resistance for these structures may be more critical than structural stiffness or strength.

Fatigue cracking has been the single largest cause of the damage requiring repair (forming 31.3% of the total damage) to the offshore platforms in the North Sea

(Ramachandra Murthy 1998). The primary reason for the collapse of the accommodation platform 'Alexander L. Kielland' in the year 1980 in the North Sea, which killed 212 men on board, was the failure of a brace due to fatigue cracking. More than 75% failures in steel structures, especially in welds or welded structures, are attributed directly or indirectly due to fatigue cracking (Ramachandra Murthy & Seetharaman 1997).

Fatigue failure is progressive in nature and involves the following four stages: (1) crack initiation at the points of concentrated stress, (2) crack growth, (3) crack propagation, and (4) final rupture. The failure begins with a slip in the crystalline structure of the metal followed by the development of microscopic cracks, which gradually increase in size in a slow and steady mode. (Crack initiation always occurs in points of flaw in the material or stress concentrations.) This progressive cracking usually results in 'striation' marks (also called *clam shell* or *oyster shell* marks), which are concentric rings that point toward the area of the initiation (Gaylord et al. 1992; Lu & Mäkeläinen 2003). The cracked surface with striations is a clear indication of the fatigue crack (see Fig. 17.1). Although somewhat similar in appearance, striations are not beach marks as one beach mark may contain thousands of striations. The extent of the fatigue crack gradually increases with the subsequent load repetitions. The fracture line will not be easily visible on the surface of the member in service conditions but can be detected with non-destructive testing techniques. The final (catastrophic) failure generally occurs in regions of tensile stress when the reduced cross section becomes insufficient to carry the peak load without rupture. Fatigue damage of structures subject to elastic stress fluctuations occurs at regions where the localized stress exceeds the yield stress of the material**.** After a certain number of load fluctuations, the accumulated damage causes the initiation and subsequent propagation of crack(s) in the plastically damaged regions. This process can cause the components fracture. The usual appearance of the face of fatigue fracture is shown in Fig. 17.2 (Clarke & Coverman

Fig. 17.1 Typical striations around an inclusion (Lu & Mäkeläinen 2003)

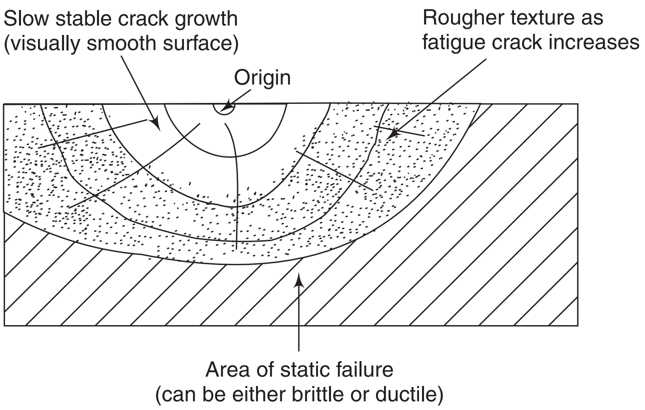


Fig. 17.2 Appearance of fatigue fracture surface (Clarke & Coverman 1987)

1987). Another possible mode of failure is that the fracture limit state is reached (brittle fracture) after a fatigue area has grown to a critical size (Kulak & Gilmor 1998). The large final fracture area for a given material indicates a high maximum load, whereas a small area indicates a low fracture load (Lu & Mäkeläinen 2003).

Fatigue failures may be classified as high-cycle and low-cycle fatigue failures. Under high-cycle fatigue, the material deforms primarily elastically, and the number of cycles for failure, or the failure time, is characterized in terms of the stress range. Low-cycle fatigue can be characterized by the presence of macroscopic cyclic plastic strains as evidenced by a stress–strain hysteresis loop. Depending on the material strength and ductility, the upper limit of the low-cycle fatigue regime may be from 100 to 100,000 cycles or more. For common ductile structural materials, the low-cycle fatigue regime is generally limited to less than 50,000 cycles.

Source of Flaws

As already discussed, the crack initiation is due to the effect of stress concentrations or flaw(s) in the material. Stress concentrations arise due to sudden changes in the general geometry (e.g., 'notch' intersection of two elements) of a member and from local changes due to bolt and rivet holes. Stress concentration also occurs at defects in the member or its connectors and welds. These may be due to the original rolling of steel or subsequent fabrication such as poorly made welds, rough edges resulting from shearing, punching, or flame cutting.

Flaws in rolled shapes arise from surface and edge imperfection: irregularities in mill scale, laminations, seams, inclusions, etc. due to material handling, cutting, shearing, etc. (Ramachandra Murthy & Seetharaman 1997).

Punched holes give a greater reduction in fatigue life than drilled and reamed holes because of the imperfections at the hole edges arising from the punching process. In any case, the crack starts at the edge of the hole. Such flaws may not be noticed during fabrication or erection but, as discussed already, they give rise to cracks under cyclic loading.

Defects in Welded Members

In welded steel structures most of the fatigue cracks start to grow from welds, rather than from other details, because of the following points (Gurney 1979; Fisher 1984; Maddox 2005).

1. Most welding processes leave minute metallurgical discontinuities, from which cracks may grow. As a result, the initiation period needed to start a crack in the material is either very short or not existent. Cracks, therefore, spend most of their life propagating, i.e., getting longer.
2. Most structural welds have a rough profile. Sharp changes in direction generally occur at the toes of butt welds and at the toes and roots of fillet welds (see Fig. 17.3). These points cause local stress concentrations (see Fig. 17.4). Small discontinuities close to these points will therefore react as though they are in a more highly stressed member and grow faster.

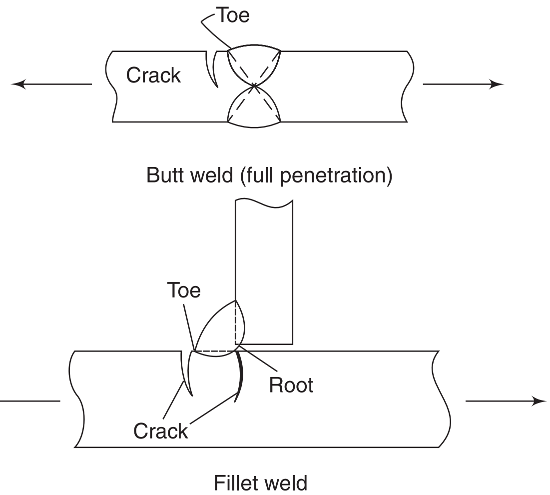


Fig. 17.3 Local stress concentration at welds

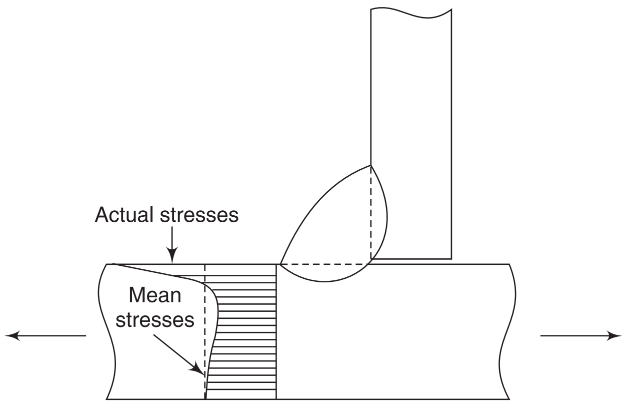


Fig. 17.4 Typical stress distribution at weld toe

Imperfection in welds include the following (Gurney 1979; Barsom & Rolfe 1999):

- Volumetric (blow holes and pores and slag inclusion) imperfections
- Planer imperfections (cracks and lack of fusion)
- Imperfections introduced due to the weld geometry (weld reinforcement, inclusion around a weld repair or at start–stop locations or at arc strikes and undercut)
- Imperfection of the weld geometry (angular and axial misalignment)

Some of these imperfections are shown in Fig. 17.5.

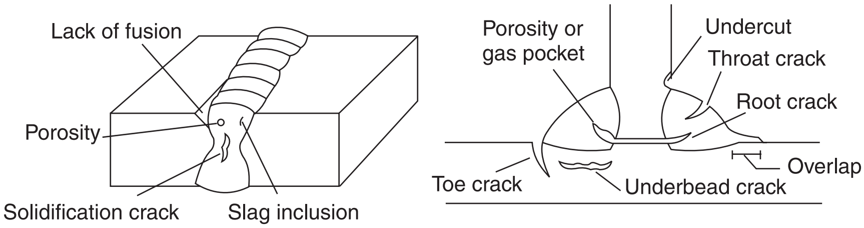


Fig. 17.5 Imperfections in welds

Imperfections lower the fatigue strength and are caused by the following processes (Barsom and Rolfe 1999):

- Improper design that restricts accessibility for welding
- Improper choice of the electrode and flux
- Incorrect selection of the welding process
- Human errors such as welder's negligence

Although the fabricator of the structure and the weld inspector (who certifies the quality of welds) may try to minimize these defects, it is neither practical nor economically feasible to eliminate all these imperfections. Thus, the potential for *fatigue crack growth* exists in every steel structure subjected to alternating loads. Also, the initiation phase of fatigue crack growth does not exist in most welded structures, since the crack is available already in the form of an existing flaw or discontinuity. However, fortunately the crack growth requires a relatively large number of cycles of loading.

As discussed in Chapter 2, the fracture due to fatigue is an ultimate (safety) limit state and the fatigue strength is always checked under service-load conditions. Hence the task of the structural engineer is to proportion the structural members (that have the potential for failure by fatigue crack growth) such that they will have long life compared to the design life of the structure.

17.1 Factors Affecting Fatigue Life

Of the many factors that affect the fatigue life, the following three factors are found to be very important.

1. The *number of cycles* of loading to which the member is subjected to.
2. The *stress range* at the location (the algebraic difference between the maximum nominal stress and the minimum nominal stress. The nominal stresses are those

determined from the applied loading and using the cross-sectional properties of the members using the basic theory of the strength of materials).

3. The type of member/detail at the location.

The other factors that influence the fatigue behaviour are (a) stress concentration, (b) residual stress in the steel, (c) plate thickness, (d) material strength, (e) imperfections, (f) stress ratio, (g) frequency of cyclic loading, (h) post-weld treatment, (i) service temperature of the steel, and (j) environment (important in the case of corrosion fatigue). Some of these factors are discussed briefly in the following.

Stress concentration Fatigue failure is often enhanced by the stress concentration at the tip of flaws such as holes, notches, etc. as shown in Fig. 17.6. Notches cannot be avoided in structures and machines. A bolt has notches in thread roots and at the transition between the head and the shank. The stress at these points will be more than three times the average applied stress. When the load is cyclic, the stress fluctuates between tension and compression and points *m* and *n* (see Fig. 17.6) experience a higher range of stress reversal than the applied average stress. Due to the fluctuations involving higher stress ranges, minute cracks appear at these points (Fig. 17.7). The final fracture occurs when the crack length reaches a critical value

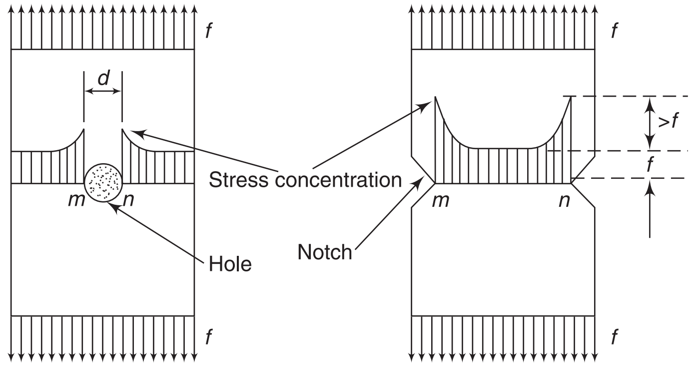


Fig. 17.6 Stress concentrations in the presence of notches and holes

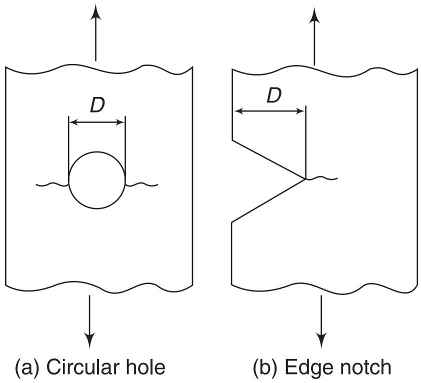


Fig. 17.7 Fatigue cracks at holes and notches

a_e or the stress intensity factor K_I reaches the critical value K_{IC} (which is termed as *fracture toughness*).

The fracture toughness can be determined experimentally from tests with a predetermined crack of length a. K_{IC} assumes the fracture without plastic deformation. For relatively thin plates, larger plastic deformations occur and the critical intensity factor is then defined as K_C, which is higher than K_{IC}.

Any abrupt discontinuity or change in the section of a member, such as a sharp re-entrant corner, interrupts the transmission of stress along smooth lines (see Fig. 17.8). The magnitude of the stress concentration increases as the abruptness of the discontinuity increases (Lu & Mäkeläinen 2003). These localized concentrations, including those at the toe of longitudinal welds, have significant effect on fatigue strength.

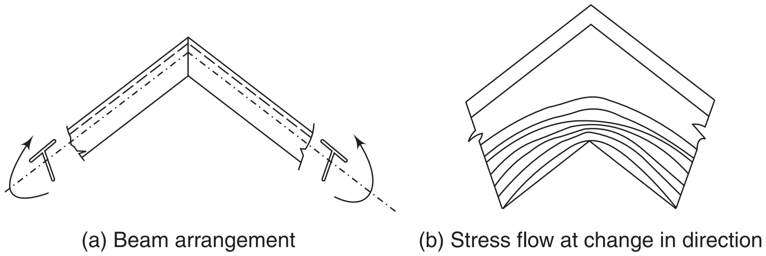


Fig. 17.8 Stress flow at re-entrant corners

Residual stresses As discussed in Chapter 1, residual stresses are those that exist in a component that is free from externally applied loads. They are caused by non-uniform plastic deformations in neighbouring regions. Furthermore, residual stresses are always balanced so that the stress field is in static equilibrium. Because the fatigue life is governed by the stress range instead of stresses, tensile residual stresses usually have only a secondary effect on the fatigue behaviour of components. On the other hand, excessive tensile residual stress can also initiate an unstable fracture in materials with a low-fracture toughness. The effect of compressive residual stress may increase the fatigue resistance for a lower level of stress.

In welded components, residual stresses are caused by the inability of the deposited molten weld metal to shrink freely as it cools and solidifies. The magnitude of the residual stresses depends on such factors as the deposited weld beads, weld sequence, total volume of deposited weld metal, weld geometry, and the strength of the deposited weld metal with respect to the adjoining base metal. Often the magnitude of these stresses exceeds the elastic limit of the lowest strength region in the weldment (Barsom & Rolfe 1999).

Plate thickness The thickness of the plate has an adverse effect on the fatigue strength due to the following reasons (Fricke 2003).

Stress gradient effect The tensile region of the stress field (including residual stresses) around the weld toe is larger in thicker plates so that an initial defect experiences a larger stress during crack initiation and early crack propagation, thus resulting in a shorter fatigue life.

Technological size effect This effect is mainly attributed to the material size and surface effects. In particular, for welded joints, the ratio between the plate thickness and the weld toe radius is larger for thicker plates, resulting in a higher stress concentration and, hence, in a reduced crack initiation period.
Statistical size effect The likelihood of finding a significant defect in a larger volume is increased compared to a smaller one.

Material strength The grade of steel has no apparent effect on the number of cycles to failure. In general, the fatigue resistance of steel is proportional to its ultimate strength. Under ideal conditions, the fatigue limit is approximately 35%–50% of the ultimate tensile strength for most steels and copper alloys. The general relation between the fatigue limit (f_l) and the ultimate stress (f_u) is given by (Ramachandra Murthy 1998)

$$f_l = 140 + 0.25 f_u \quad \text{(in MPa)} \tag{17.1}$$

Imperfections Normally the imperfections and flaws discussed in the previous sections can cause stress concentration that lowers the fatigue strength. The severity of a discontinuity, which is due to imperfections, is governed by its size, shape, and orientation, as well as the magnitude and direction of the design and fabrication stresses. Generally, the severity of discontinuity increases as the size increases and as the geometry becomes more planar and the orientation more perpendicular to the direction of tensile stresses. Thus volumetric discontinuities are usually less injurious than planar, crack-like discontinuities. Also the crack-like discontinuities whose orientation is perpendicular to the tensile stress can be injurious than those parallel to the tensile stress. Furthermore, a surface discontinuity whose plane is perpendicular to the tensile stress is more severe than if it were embedded (Barsom & Rolfe 1999).

In partial penetration fillet-welded connections, a crack or gap is left due to the lack of penetration. Since this crack will be parallel to the direction of the bending stress in beams, the gap will not open up under the application of stress. Similarly, consider an I-shaped beam with a cover plate fastened to the beam flanges by bolts. Now the region between bolt lines constitutes a ‘flaw’ or ‘crack’. But, since it is parallel to the stress field, it will not affect the strength of the member. Broadly speaking, any mechanical detail with bolts has a better fatigue life than does its equivalent welded detail. Moreover, the inspection for defects in welding is more difficult than in fastened detail. Similarly, the repair of a welded joint is more difficult than a fastened joint.

Stress ratio and frequency of cyclic loading From a large amount of test specimens, it has been found that the stress ratio R has little influence on the fatigue behaviour of welded components (Fricke 2003). This is so because tensile residual stresses up to yielding are expected at the critical crack initiation points of the welded structures. Then the stress cycles remain tensile, irrespective of the R values of the external load. Therefore, the influence of the stress ratio is only taken into account very cautiously or not at all in the codes or regulations (Lu & Mäkeläinen 2003). The fatigue limit (see Section 17.4.3) for $R = -1$ is found to be about one-half of the tensile strength of steel (Gaylord et al. 1992). The effect of minimum stress (due to dead loads) is considered negligible for design purposes. The frequency of loading also does not influence the fatigue strength when the applied stress range is low and the frequency is less than 50 Hz. However, when the stress range is high, an increase in frequency may increase the fatigue strength.

Influence of post-weld treatment Using the post-weld treatment of the weld, it is possible to improve the fatigue strength of welded joints considerably, especially the fatigue limit. The improvement mainly involves an extension of the crack initiation life and can be achieved by the following measures (Fricke 2003):

- A reduction in the stress peak related to the weld shape
- Removal of crack-like weld imperfections at the weld toe
- Removal of detrimental tensile residual stresses, up to the formation of favourable compressive residual stresses in the area susceptible to crack initiation

Post-weld treatment is of particular interest in connection with the repair of fatigue cracks. However, it must be guaranteed that the fatigue strength of the area which is not subjected to post-weld treatment is high enough.

17.2 Different Approaches to Fatigue Analysis

The first known investigators concerned with fatigue phenomena were designers of axles for locomotives. Wöhler's experiments with axles during 1852–60 were the first known laboratory tests with the objective to derive and quantitatively describe the limits for fatigue. This was followed by more elaborate analyses of stresses and their effects on fatigue by Berber, Goodman (during 1899), and others. Prof. August Thum and Prof. Ernst Gassner of the University of Darmstadt coined the word *Betriebsfestigkeit* (structural stability) in 1941 and formulated a procedure for the experimental simulation of variable amplitude loading. Prof. Kurt Kloeppel in 1950s published allowable stresses for design details used in overhead cranes and railway bridges. The development of theories on the effects of plastic deformation on fatigue resulted in the strain method discovered by Manson and Coffin in 1954. The theory of crack propagation was started by Griffith and continued by Paris and others in 1961. Fatigue was incorporated into design criteria towards the end of the nineteenth century and has been extensively studied since then (Lu & Mäkeläinen 2003).

According to the definition of the fatigue life, the approaches for fatigue analysis can be classified into (1) the stress method, (2) the strain method, and (3) the crack propagation method.

Stress and *strain methods* characterize the total fatigue life in terms of cyclic stress range or strain range. In these methods, the number of stress or strain cycles to induce fatigue failure in an initially uncracked or smooth surfaced laboratory specimen is estimated under controlled cyclic stress or strain. The resulting fatigue life includes the fatigue crack initiation life to initiate a dominant crack and a propagation of this crack until catastrophic failure. Normally, the fatigue initiation life is about 90% of the total life due to the smooth surface of the specimen (Suresh 1998). Under a high-cycle (> 10^2 to 10^4) low-stress fatigue situation, the material deforms primarily elastically and the failure time has traditionally been described in terms of stress range. However, stresses associated with low-cycle fatigue (< 10^2 to 10^4) are generally high enough to cause plastic deformation prior to failure. Under these circumstances, the fatigue life is described in terms of the strain range. The low-cycle approach to fatigue design has found particularly widespread use in ground vehicle industries (Socie & Marquis 2000).

The basic premise of *crack propagation method* (fracture mechanics approach) is that all engineering components are inherently flawed. The size of a pre-existing flaw is generally determined from non-destructive flaw detection techniques, such as visual, dye-penetrant, or x-ray techniques, or the ultrasonic, magnetic, or acoustic emission methods. The fatigue life is then defined as the number of cycles to propagate the initial crack size to a critical size. The choice of the critical size of cracks may be based on the fracture toughness of the material, the limit load for the particular structural part, the allowable strain, or the permissible change in the compliance of the component. The prediction of fatigue life is mainly based on linear elastic *fracture mechanics*. The crack propagation method, which is a conservative approach to fatigue, has been widely used in fatigue-critical applications where catastrophic failures will result in the loss of human lives, such as aerospace and nuclear industries (Socie & Marquis 2000).

The strain method and the crack propagation method are outside the scope of this book and we will confine our attention to the stress method, which is followed by the IS: 800. Interested readers may consult literature Lu and Mäkeläinen (2003), Gurney (1979), Fricke (2003), and Suresh (1998) for the details of other methods. Before discussing the stress method using *S-N* curves, let us discuss the parameters involved in constant-amplitude fatigue loading.

17.3 Fatigue Loading

Structural components are subjected to two kinds of load history in fatigue design. The simplest load history, which occurs in machinery parts such as shafts and rods during periods of steady-state rotation is the constant-amplitude loading (see Fig. 17.9) and has the following parameters.

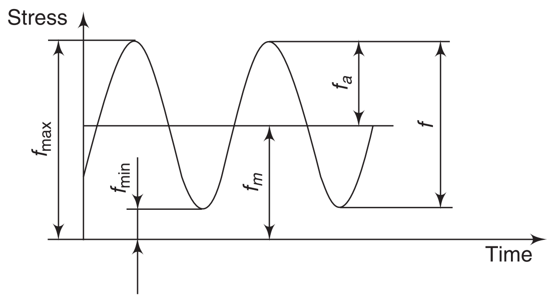


Fig. 17.9 Terminology used in constant amplitude loading

1. *Applied stress range f*, which is the algebraic difference between the maximum stress f_{max} and the minimum stress f_{min} in the cycle, i.e.,

$$f = f_{max} - f_{min} \tag{17.2}$$

2. *Mean stress*, which is the algebraic mean of f_{max} and f_{min} in the cycle, i.e.,

$$f_m = (f_{max} + f_{min})/2 \tag{17.3}$$

3. *Stress amplitude*, which is half the stress range in a cycle, i.e.,

$$f_a = (f_{max} - f_{min})/2 \tag{17.4}$$

4. *Stress ratio*, which represents the relative magnitude of the minimum and the maximum stress in a cycle, i.e.,

$$R = f_{min}/f_{max} \tag{17.5}$$

The values of R corresponding to various loading cases are shown in Fig. 17.10. Thus $R = 0$ denotes stress ranging from zero to tension; $R = -½$ denotes stress alternating between tension and compression equal to half the tension, etc. The stress fluctuation from a given minimum tensile load to a maximum tensile load is characterized by a positive value between 0 and 1 ($0 < R < 1$).

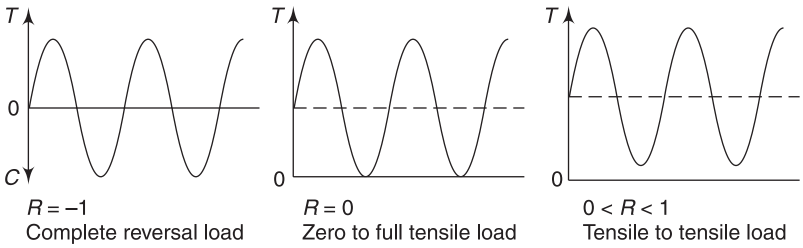


Fig. 17.10 *R*-ratios for various loadings

The second load history is the variable-amplitude loading history, in which the probability of the same sequence and magnitude of stress ranges recurring during a particular time interval is very small and cannot be represented by an analytical function (see Fig. 17.11). This type of loading is experienced by many structures, such as wind loading on aircraft, wave loading on ships and offshore platforms, and truck loading on bridges.

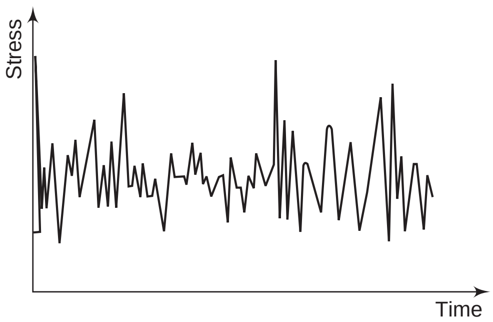


Fig. 17.11 Variable-amplitude loading history

Either constant-amplitude loading or variable-amplitude loading can cause unidirectional stresses in structural components, such as pure tension and compression, pure bending, or pure torsion. For the components with complicated geometries the resulting stresses may be in different directions. We are considering only unidirectional cases in this book. The effect of multi-axial loading may be found in the work by Socie and Marquis (2000).

17.4 Stress Methods

In the case of static loading, the yield strength or ultimate strength of the material is obtained from tensile loading. The structural members are designed according to these values. Similarly, under the fluctuating loads, the significant strength is the *fatigue strength* or *fatigue limit*. The fatigue strength is defined as the range of cyclic stress a material can withstand for a given number of cycles without causing any failure (see Fig. 17.12). The fatigue limit or *endurance limit* is defined as the maximum value of the stress range that can be repeated an infinite number of times on a test specimen without causing failure (see Fig. 17.12). It is generally considered to correspond to a fatigue life of about 2 to 5 million cycles for ferrous alloys.

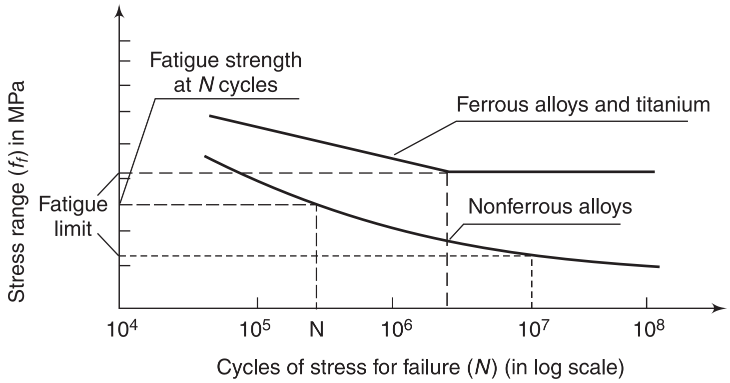


Fig. 17.12 Typical *S-N* curves for ferrous and non-ferrous alloys

In stress methods, it is necessary to determine the fatigue strength and/or fatigue limit (analogous to yield strength) for the material so that cyclic stresses can be kept below that level to avoid fatigue failure for the required number of cycles. The structural components are designed in such a way that the maximum stress never exceeds the materials fatigue strength or fatigue limit. Thus the stresses and strains remain in the elastic region such that no local yielding occurs to initiate a crack.

17.4.1 *S-N* Curves

The most common way to describe the fatigue testing data is using *S-N* curves that show the relationship between the number of cycles, N, for fracture and the maximum stress, S, or stress range, f_f, due to the applied cyclic load. Generally, the abscissa is the logarithm of N and the ordinate may be S, the stress or stress range f_f or the logarithm of stress or stress range.

The stress to be calculated is simply the nominal stress at the location of the detail (which can be determined from the applied loading such as forces and moments and the cross-sectional properties of a component, based on the basic theory of the strength of materials). This simple representation of stress is possible because we can include the effect of stress concentration in the selection of detail itself.

Another approach is the *notch stress method* in which the local maximum stress due to stress risers is calculated using the notch factor and nominal stress (Lu & Mäkeläinen 2003). This method requires the knowledge of the stress distribution in the vicinity of the weld, which is usually obtained by means of a finite element analysis. More discussions on the notch stress method could be found in the literature by Fricke (2003) and von Wingerde et al. (1995).

Since fabricated steel structures always contain a high magnitude of residual stresses, it is always preferable to use stress range, rather than the maximum stress or stress ratio, as the parameter to indicate the failure.

A typical standard S-N curve is shown in Fig. 17.12. Most of the fatigue tests are performed in the high-cycle fatigue domain, where a linear relationship between the stress range and the fatigue life exists in a log-log diagram. This linear relationship can be expressed as

$$\log N = \log C - m \log f_f \tag{17.6}$$

where N is the number of cycles to failure, C is a constant, f_f is the fatigue strength dependent on detail category, and m is the slope of the fatigue strength curve with a value of 3 and/or 5.

Alternatively, Eqn (17.6) can be written as

$$N = N_R(f_f/f_R)^{-m} \tag{17.7}$$

where f_f is the fatigue strength at loading cycle N (dependent on detail category), f_R is the characteristic value of the fatigue strength at loading cycle N_R ($N_R = 2 \times 10^6$ or 5×10^6), and m is the slope of the fatigue strength curve.

Equation (17.7) can be rearranged as per the equation given in the code as

$$f_f = f_R(N_R/N)^{1/m} \tag{17.8}$$

In order to obtain the meaningful engineering data, a large amount of testing should be carried out. However, even though the same specimen is used in the fatigue tests, the results show a wide range of dispersion. This is due to the different geometrical micro-irregularities of surfaces for the same type of specimen. These different local concentrations cause different fatigue lives. Therefore, it is necessary to carry out the statistical analysis of fatigue data. This in turn makes it necessary to consider the effect of failure probability. The curves formed by integrating the failure probability into the S-N curve are called *P-S-N curves* (Lu & Mäkeläinen 2003). The standard S-N curve provided in codes corresponds to a 50% probability of failure, i.e., $P = 0.5$ (Lu & Mäkeläinen 2003).

The main objective of the fatigue analysis is to design the structural components for an infinite life or for a limited life. The objective of the infinite life design is to ensure that the working stress due to loading is below the fatigue limit. Whereas the objective of the limited life design is to predict the number of cycles available within the fatigue life based on the stress level, or conversely to determine the stresses on a given number of cycles (Gaylord et al. 1992). It is seen from Fig. 17.12 that the fatigue life reduces with increase in the stress range and at a limiting value of stress, called *fatigue limit* or *endurance limit*, the curves flattens out.

For normal steel and many ferrous alloys, the 'knee' point of the fatigue limit is normally in the range of 10^5–10^7 cycles after which failure does not occur. Many high-strength steels, aluminum alloys, and other materials do not generally exhibit a knee point of the fatigue limit. For these materials, the fatigue limit is defined at the stress level corresponding to 10^7 cycles (Suresh 1998).

It has to be noted that the following factors affect the *S-N* curve: change in mechanical properties or microstructure of the material, chemical environment, cyclic frequency, temperature, residual stress, and surface effects (Lu & Mäkeläinen 2003; Maddox 2005).

17.4.2 Safe-life and Fail-safe Concepts of Structural Design

'Fail-safe' and 'safe life' are the two concepts of fatigue-resistant structural design (Lu & Mäkeläinen 2003). In the *safe-life method*, the designer starts by making an estimation of the load spectrum to which the critical structural components are likely to be subjected in service. These components are then analysed or tested under that load spectrum to obtain their expected life. Finally a factor of safety is applied in order to give a safe life during which the possibility of a failure due to fatigue is considered to be sufficiently remote. It is clear that by making the safety factor sufficiently large, the designer can govern the probability of the failure associated with his/her design. On the other hand, if a fatigue crack does occur, it may well be catastrophic, and safety depends on achieving a specified life without a fatigue crack developing. In this method, the emphasis is on preventing the crack initiation (Lu & Mäkeläinen 2003).

In the *fail-safe concept*, the basis of design is that even if a crack in the main member does occur, there will always be sufficient strength and stiffness in the remaining members to enable the structure to be used safely. This concept implies that periodic in-service inspection is a necessity, and that the methods used must be such as to ensure that cracked members are discovered so that repairs or replacements can be made.

It is clear that with the fail-safe method of design the probability of partial failure is much greater than with the safe-life design. In developing a fail-safe structure, the safe life should also be evaluated, in order to make sure that it is of the right order of magnitude. However, the emphasis, instead of being on the prevention of crack initiation, is on producing a structure in which a crack will propagate slowly and which is capable of supporting the full design load after partial failure. The basic principle of the fail-safe design is therefore to produce a multiple load-path structure and preferably a structure containing crack arresters. In addition, the structural elements must be arranged so as to make inspections as easy as possible. In areas where this is not possible, the elements must be oversized so that either fatigue cracking does not occur in them or the fatigue crack growth is so slow that there is no risk of failure (Lu & Mäkeläinen 2003).

17.4.3 Using *S-N* Methods to Evaluate Fatigue Life

Several *S-N* methods are available for estimating the fatigue life of welded components: nominal stress method, structural hot spot stress method, notch stress method, notch stress intensity method, and notch strain method (Fricke 2003). Fatigue assessment according to the nominal stress method uses several *S-N* curves together with detail classes of basic joints. This is the simplest and most common method adopted for estimating the fatigue life of structural joints and elements. The Eurocode 3-1993, Canadian code CAN/CSA-S.16.1, 2001, and the Indian code IS: 800 are based on this method.

The fatigue strength in IS: 800 is defined by a series of log f_f – log N or log τ_f – log N curves (see Figs 17.13 and 17.14), each applying to a typical detail category. Each category is designated by a number which represents the reference value f_{fn} (normal fatigue stress range) at 2 million cycles, i.e., the number of stress cycles, $N_{sc} = 2 \times 10^6$. The values are rounded values. Some common detail types and their fatigue categories are provided in Table 13.3(1) to Table 13.3(4) of the code. Figure 17.14 shows a few detail category classifications as per IS: 800. Two other concepts are depicted in Fig. 17.13. One is the constant-amplitude fatigue limit f_d, which is the limiting stress range value above which a fatigue assessment is necessary. The number of cycles corresponding to constant amplitude fatigue

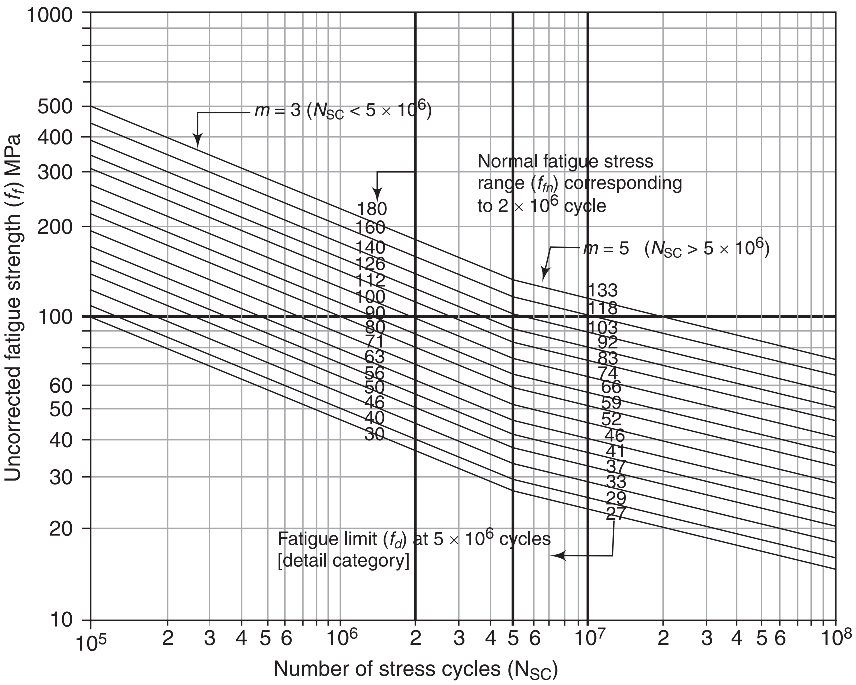


Fig. 17.13 *S-N* curves for normal stress as per IS: 800

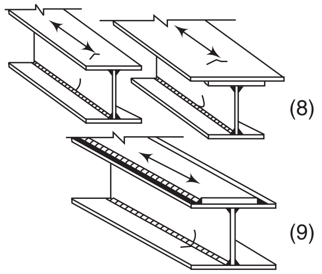
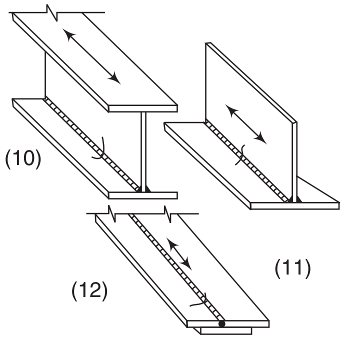
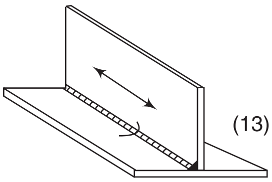
Detail category	Constructional details	
	Illustration (see note)	**Description**
92		**Welded plate I-section and box girders with continuous longitudinal welds** (8) & (9) zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having un-repaired stop–start positions.
83		**Welded plate I-section and box girders with continuous longitudinal welds** (10) & (11) zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having un-repaired stop–start positions. (12) Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop–start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 92.
66		**Welded plate I-section and box girders with continuous longitudinal welds** (13) Zones of continuous longitudinal welds carried out from one side only, with or without stop–start positions.

Fig. 17.14 Detail category classifications as per IS 800 : 2007

limit is 5 million cycles, i.e., $N_d = 5 \times 10^6$. The other is cut-off limit, f_L, which is a limit below which stress ranges of the design spectrum do not contribute to the calculated cumulative damage. The number of cycles corresponding to this value is 10^8 cycles, i.e., $N_L = 10^8$. The cut-off limit is specified because when variable-amplitude loading is applied, the stresses less than the fatigue limit still cause damage due to the fact that larger amplitude cycles may start to propagate the crack.

The fatigue strength curves for nominal stresses are defined by Eqn (17.6) or Eqn (17.8). The constant representing the slope of the fatigue strength curves, m, is taken as 3 and/or 5, and $\log C$ is a constant that depends on the related part of the slope. Their values are given in Table 17.1. Similar fatigue strength curves are used for shear stresses (Fig. 17.15) and only one slope value is taken, i.e., $m = 5$.

Table 17.1 Numerical values for fatigue strength curves

	log C for $N < 10^8$		Stress range at constant-amplitude fatigue limit	Stress range at cut-off limit
Normal stress range				
f_{fn} (N/mm^2)	$N \leq 5 \times 10^6$ ($m = 3$)	$N > 5 \times 10^6$ ($m = 5$)	f_d (N/mm^2)	f_L (N/mm^2)
160	12.901	17.036	118	64
140	12.751	16.786	103	57
125	12.601	16.536	92	51
112	12.451	16.286	83	45
100	12.301	16.036	74	40
90	12.151	15.786	66	36
80	12.001	15.536	59	32
71	11.851	15.286	52	29
63	11.701	15.036	46	26
56	11.551	14.786	41	23
50	11.401	14.536	37	20
45	11.251	14.286	33	18
40	11.101	14.036	29	16
36	10.951	13.786	27	14
Shear stress range				
τ_{fn} (N/mm^2)	$N < 10^8$ ($m = 5$)		τ_d (N/mm^2)	τ_L (N/mm^2)
100	16.301		83	46
80	15.801		67	36

Source: (Lu & Mäkeläinen 2003; Eurocode 3, 1992)

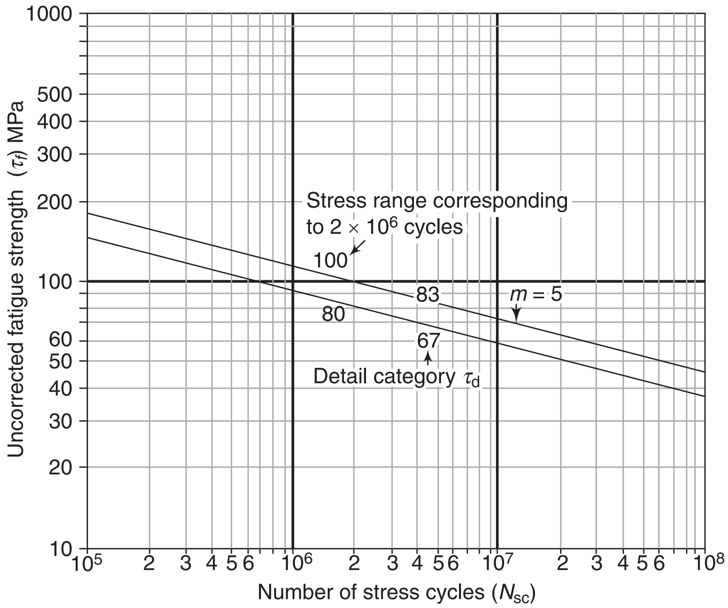


Fig. 17.15 *S–N* curves for shear stress as per IS 800 : 2007

These curves are based on representative experimental investigations and thus include the effects of local stress concentrations due to the weld geometry, size and shape of acceptable discontinuities, the stress direction, residual stresses, metallurgical conditions, and, in some cases, the welding process and the post-weld improvement procedures (Lu & Mäkeläinen 2003).

No fatigue assessment is required when any of the following conditions is satisfied according to the code:

1. The largest nominal stress range $f_{f\,\max}$ satisfies

   $$f_{f\,\max} \le 27\mu_c/\gamma_{\text{mft}} \tag{17.9}$$

   where μ_c is a correction factor [see Eqn (17.12)] and γ_{mft} is the partial safety factor for fatigue strength (see Table 17.2)

2. The highest shear stress range $\tau_{\max}$ satisfies

   $$\tau_{f\,\max} \le 67\mu_c/\gamma_{\text{mft}} \tag{17.10}$$

3. The total number of actual stress cycles, N_{sc}, satisfies

   $$N_{\text{sc}} \le 5 \times 10^6\,[27\mu_c/(\gamma_{\text{mft}}\,f_{f\text{eq}})]^3 \tag{17.11}$$

In these conditions, $f_{f\text{eq}}$ is the equivalent constant-amplitude stress range (MPa), which is defined as the constant-amplitude stress range that would result in the same fatigue life as for the spectrum of variable-amplitude stress ranges, when the comparison is based on Miner's summation (see Section 17.5). γ_{mft} is the partial safety factor for fatigue loading and its value has to be taken as per Table 17.2.

Table 17.2 Partial safety factors for fatigue strength (γ_{mft})

Inspection and access	Consequence of failure	
	Fail-safe	Non-fail-safe
Periodic inspection and maintenance, accessibility to detail is good	1.00	1.25
Periodic inspection and maintenance, poor accessibility for detail	1.15	1.35

The code also recommends a partial safety factor for loads on the evaluation of the stress range in fatigue design as 1.0. The design stress should be determined by the elastic analysis of the structures by using conventional stress analysis methods. The normal and shear stresses should be determined considering all design actions on the members, but excluding the stress concentration due to the geometry of the detail. The code stipulates that the stress concentration not characteristic of the detail should however be accommodated in the stress calculation.

In the fatigue design of trusses made of members with open cross sections and not having pinned ends, the stresses due to secondary bending moments should be taken into account unless the slenderness ratio (KL/r) of the member is less than 40.

To take into account the influence of the thickness of the plate, the code gives a capacity reduction factor μ_c which should be applied when plates of thicknesses more than 25 mm are joined together by transverse fillet welds:

$$\mu_c = (25/t_p)^{0.25} \le 1.0 \tag{17.12}$$

where t_p is the actual thickness (in mm) of the thicker plate being jointed. This reduction should be applied only to structural details with welds transverse to the direction of normal stresses (Eurocode 3 1992). Also when the detail category in the classification tables of the code already varies with thickness, the above correction for thickness should not be applied (Eurocode 3 1992).

The fatigue strength of the standard detail of the normal fatigue range not corrected for the effects discussed above is given by the code as below:

When $N_{sc} \leq 5 \times 10^6$,

$$f_f = f_{fn} \sqrt[3]{5 \times 10^6 / N_{sc}} \tag{17.13}$$

when $5 \times 10^6 \leq N_{sc} \leq 10^8$,

$$f_f = f_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}} \tag{17.14}$$

Similarly, the shear fatigue stress range is given by the code as

$$\tau_f = \tau_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}} \tag{17.15}$$

where f_f and τ_f are the design normal and shear fatigue stress ranges of the detail, respectively, for life cycle of N_{sc} and f_{fn} and τ_{fn} are the normal and shear fatigue strengths of the detail for 5×10^6 cycles. At any point in the structure, if the actual normal and shear stress ranges, f and τ, are less than the design fatigue strength range corresponding to 5×10^6 cycles, with an appropriate safety factor, no further assessment for fatigue is necessary at that point.

It has to be noted that the absolute maximum value of the normal and shear stresses should never exceed the elastic limit (f_y, τ_y) for the material under static loading. The maximum stress range shall not also exceed $1.5f_y$ for normal stresses and $1.5f_y/\sqrt{3}$ for shear stresses under any circumstances as per the code. The code also states that the actual normal and shear stress range, f and τ, at a point of the structure subjected to N_{sc} cycles in life shall satisfy

$$f \leq f_{fd} = \mu_c f_f / \gamma_{mft} \tag{17.16}$$

and

$$\tau \leq \tau_{fd} = \mu_c \tau_f / \gamma_{mft}$$

where μ_c is the correction factor defined by Eqn (17.12), γ_{mft} is the partial safety factor against fatigue failure as given in Table 17.2, and f_f and τ_f are the normal and shear fatigue ranges for the actual life cycle N_{sc} as given by Eqns (17.13)–(17.15).

17.4.4 Modified Goodman Diagram

As we have seen in Figs 17.13 and 17.15, the *S-N* curves are generated with fully reverse load ($R = -1$) and zero mean stresses. However non-zero mean stresses can also play an important role in resulting fatigue data. Also for the purpose of design it is more convenient to have the maximum and minimum stresses for a given life as the main parameters. The modified Goodman diagram, as shown in Fig. 17.16, provides this kind of information. This is a variation of the diagram published by Goodman in 1899. In this diagram, the maximum stresses are plotted in the vertical ordinate and minimum stresses as abscissa. Each curve in the diagram is the locus of all the points corresponding to a given fatigue life, and the diagram covers the full range of stress ratios $-1 \leq R \leq 1$. Radial lines from the origin correspond to the various stress ratios. Since $R = 1$ denotes no reversal of stress, the radial line $R = 1$ corresponds to static tension and all the curves for a given steel join at the intersection of $R = 1$ with the ordinate correspond to the tensile stress of the steel. It is also seen that the fatigue life decreases with increasing stress magnitude or stress range.

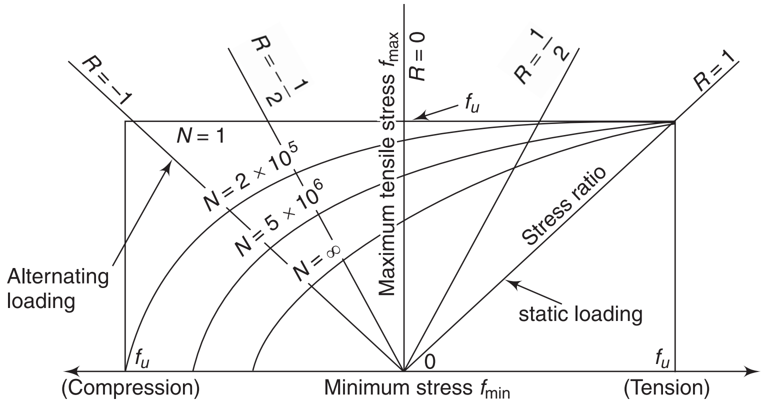


Fig. 17.16 Modified Goodman diagram showing variation of fatigue life with stress magnitude and range (Trahair et al. 2001)

Taking yield stress as the limiting useful strength, the curves in Fig. 17.16 can be represented closely by the straight lines *AB* and *BC* of Fig. 17.17. This enables fatigue–design criteria to be put in the form of simple equations. Allowable stress

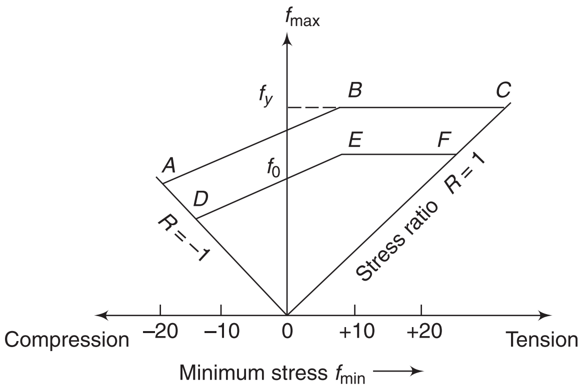


Fig. 17.17 Goodman diagram with straight line representation (Gaylord et al. 1992)

formulas may be obtained by a factor of safety to *ABC* to get *DEF*. This factor of safety can be smaller than the factor of safety for static load because of the smaller probability of the occurrence of the much larger number of cycles of the service-load magnitude needed to cause a fatigue failure (Gaylord et al. 1992).

17.5 Fatigue Assessment for Variable Stress Ranges

Until now, we discussed about fatigue properties of structural elements under a constant amplitude. But in real situations, stress ranges of different magnitudes take place at the detail and these stress ranges are applied for various lengths of time. For example, if a crane is carried by a continuous beam over several intermediate supports, more than one stress cycle will be applied per trip at a given location. This is because loading adjacent spans causes stress cycles in addition to the cycle created when the crane passes directly over the location under consideration. For this general case, one trip is termed *loading event* and the stress variation at a given point in the structure during such an event is called *stress history* (Kulak & Gilmor 1998). A state-of-the-art review on the fatigue crack growth analysis under variable-amplitude loading is available in Murthy et al. (2004). Though both linear and non-linear damage theories are available, the linear theory first proposed by Palmgren in 1924 and further developed by Miner in 1945 is used in structural engineering applications (Bannantine et al. 1990). This method is easy to understand and apply and found to give satisfactory results.

17.5.1 Palmgren–Miner Rule

The fatigue life of a component under variable-amplitude loading can be calculated using the *Palmgren–Miner rule*, which is a linear damage rule and assumes that

1. the damage fraction that results from any particular stress range level is a linear function of the number of cycles that takes place at the stress range, and
2. the total damages from all stress range levels that are applied to the detail is the sum of all such occurrences.

If n_i is the required number of cycles corresponding to the stress amplitude f_i in a sequence of m blocks and N_i is the number of cycles to failure at f_i, then the Palmgren–Miner rule states that the failure would occur when

$$\sum_{i=1}^{m} n_i/N_i \le k \tag{17.17}$$

and

$$D_i = 1/N_i \tag{17.18}$$

where D_i is called the damage of a single cycle at stress level f_i. The scheme of Plamgren–Miner's rule is shown in Fig. 17.18. The rule was first introduced by Palmgren in the analysis of ball bearings and was adapted by Miner for aircraft structure (Lu & Mäkeläinen 2003). The value of k is experimentally found to vary between 0.7 and 2.2. Usually for design purposes, k is assumed to be equal to 1.

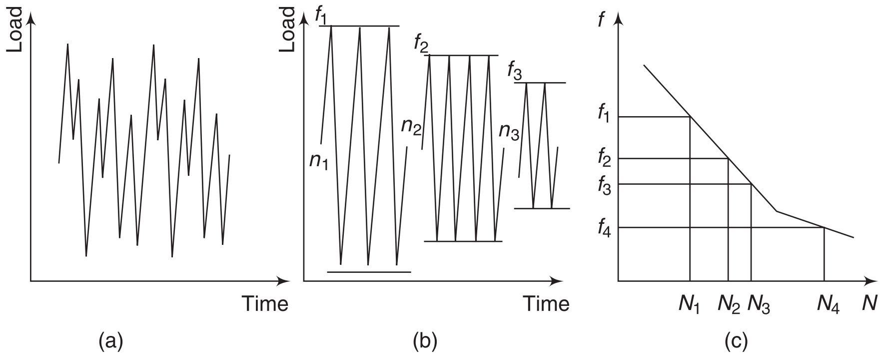


Fig. 17.18 Scheme of Palmgren–Miner's rule

The code gives the following rule based on the Palmgren–Miner rule. The fatigue assessment at any point in a structure, where in variable stress ranges f_{fi} or τ_{fi} for n_i number of cycles (i = 1 to r) are encountered, shall satisfy the following:

(a) For normal stress (f)

$$\frac{\sum_{i=1}^{\gamma_5} n_i f_i^3}{5 \times 10^6 (\mu f_{fn}/\gamma_{mft})^3} + \frac{\sum_{j=\gamma_5}^{r} n_j f_j^5}{5 \times 10^6 (\mu f_{fn}/\gamma_{mft})^5} \le 1.0 \tag{17.19}$$

(b) For shear stresses (τ)

$$\sum_{i=1}^{r} n_i \tau_{fi}^5 \le 5 \times 10^6 (\mu \tau_{fn}/\gamma_{mft})^5 \tag{17.20}$$

where γ_5 is the summation upper limit of all the normal stress ranges (f_i) having magnitudes lesser than ($\mu_c f_{fn}/\gamma_{mft}$) for that detail and the lower limit of all the normal stress ranges (f_j) having magnitudes greater than ($\mu_c f_{fn}/\gamma_{mft}$) for the detail. In the above summation all normal stress ranges (f_i and τ_i) having magnitudes less than $0.55\mu_c f_{fn}$, and $0.55\mu_c \tau_{fn}$ may be disregarded.

It should be noted that, when variable-amplitude loading is applied, the stresses lesser than the fatigue limit still cause damage due to the fact that larger amplitude cycles may start to propagate the crack. However, the linear Palmgren–Miner rule assumes independence of damage accumulation.

Empirically, tests have shown that differences exist between low–high sequences and high–low sequence. Thus, there are two main shortcomings of the linear damage rule: assuming sequence independence and damage accumulation independence. These two shortcomings might be overcome by non-linear damage rules. Also, in structural engineering applications where residual stresses are high and plasticity is restricted, the two factors were found to have small influence (Kulak & Gilmor 1998). Hence the Palmgren–Miner rule is suggested by several codes to account for cumulative damage. It is often convenient to express the Palmgren–Miner rule in terms of an equivalent stress range as follows (Kulak & Gilmor 1998):

$$f_{f\,eq} = \Sigma(n_i f_f^m / \Sigma n_i)^{1/m} \tag{17.21}$$

The code gives the following expression for the same:

$$f_{f\,\text{eq}} = \left[\frac{\sum_{i=1}^{\gamma_5} n_i f_{fi}^3 + \sum_{j=\gamma_5}^{r} n_j f_{fj}^5}{n} \right]^{1/3} \tag{17.22}$$

where $n = \sum_{i=1}^{\gamma} n_i$ (17.23)

17.5.2 Cycle Counting

When using the linear Palmgren–Miner rule to estimate the fatigue life, the variable-amplitude loading has to be transformed into a series of constant-amplitude loadings. Several methods are available to do cycle counting: reservoir counting, level crossing counting, peak counting, simple range counting, and rainflow counting. In this section, we only provide the details of *rainflow counting* (this algorithm was first devised by Tatsuo Endo and M. Matsuiski in the year 1968 and is more popular than the other methods of cycle counting). Rainflow is a generic term to describe any cycle counting method that identifies closed hysteresis loops in the stress–strain response of material subjected to cyclic loading. Several algorithms are available to perform the counting; however, they all require that the entire load history be known before the counting process starts (Downing & Socie 1982). The methodology followed in rainflow counting follows.

1. In order to eliminate the counting of half cycles, the load history has to be drawn starting and ending at the greatest magnitude.
2. The flow of rain has to be stopped when
   (a) The rain begins at a local maximum and falls opposite a local maximum that is greater than the starting point.
   (b) The rain encounters a previous flow.

Figure 17.19 illustrates the procedure of cycle counting using rainflow method. The counting is first started from the tension peaks. The details of counting based on this rule are described next.

- Route 1 starts from *A* and falls down at *B*. Since the value of *C* is less than that of *A*, the rain can continue and fall down to line *CD*. Similarly, since the value of *A* is larger than that of *E*, *C*, *I*, *K*, and *M*, it will stop at the position shown in Fig. 17.19(b). This procedure is carried out based on rule (a).
- Route 2 starts from *C* and stops as shown in Fig. 17.19(b) due to its encounter with the previous rainflow (Route 1). This illustrates rule (b).
- Route 3 starts from *E* and stops due to the value of *G* being larger than that of *E* [rule (a)].
- Route 4 is based on rule (b).
- Route 5 is based on rule (a).
- Route 6 is based on rule (b).
- Route 7 is based on rule (a).

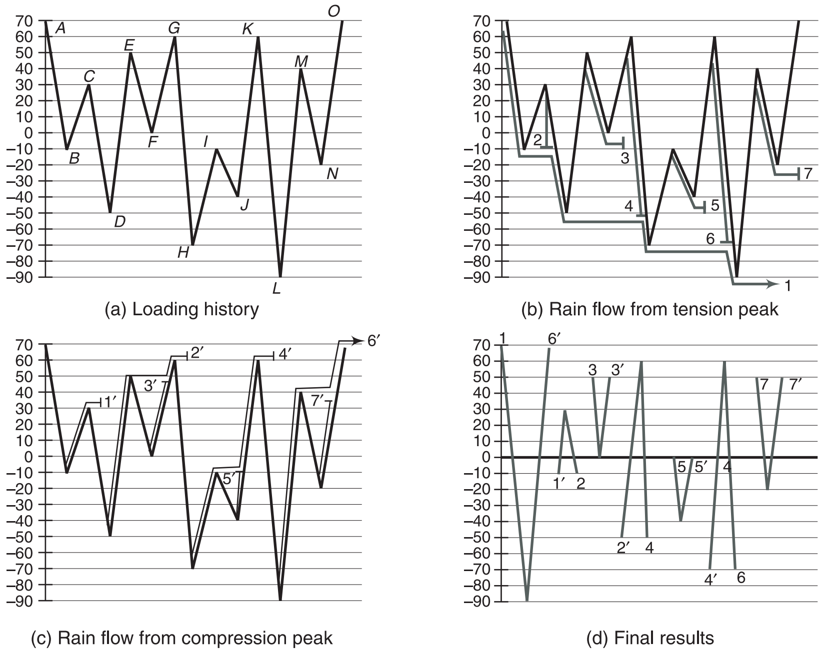


Fig. 17.19 Scheme of rainflow counting (Lu & Mäkeläinen 2003)

Similar rainflow counting from compression peaks is shown in Fig. 17.19(c). Figure 17.19(d) shows the cycles from both the tension side and the compression side. This can be done as follows.

Start from Route 1 of the tension side and find the ending point of Route 1. Then check the ratio of the compression side that starts from this point. In this case it is Route 6′. This is one cycle of loading. Similarly, other cycles in the loading history are obtained as shown in Fig. 17.19(d).

After this counting, the stress range and the number of cycles corresponding to the stress range are obtained and the damage can be estimated according to Palmgren–Miner's rule. Rainflow counting is easy to do manually for simple loading histories; however, for complex loading histories, numerical methods are to be used (Downing & Socie 1982). The details of other counting methods may be found in Gurney (1979) and Maddox (2005).

In the *reservoir method*, it is assumed that the troughs in the variable-amplitude loading history (see Fig. 17.11) act as reservoirs and for each 'peak' the lowest 'trough' drains the water. A tabular format is set up to count these reservoirs. Deducting the trough value of the stress from the peak value of the stress, the stress range in each reservoir is found. The damage resulting from each of these stress ranges is calculated using Eqns (17.6) and (17.17) and Fig. 17.13. The cumulative value of n_i/N_i suggests whether the detail is safe or not. Example 17.4 (at the end of this chapter) explains the use of the reservoir method.

17.5.3 Fatigue Assessment of Hollow Sections

The member force for hollow sections according to the code may be analysed neglecting the effect of eccentricities and joint stiffness and assuming hinged connections, provided that the effects of secondary bending moments on the stress range are considered. In the absence of a rigorous stress analysis and modelling of the joint, the effects of the secondary bending moment may be taken into account by multiplying the stress range due to the axial member forces by appropriate coefficients as given in Table 17.3 (for joints in lattice girders made from circular hollow sections) and Table 17.4 (for joints in lattice girders made from rectangular hollow sections). The values in these two tables are approximate empirical values or values based on testing.

Table 17.3 Multiplying factors for calculated stress range as per IS: 800 for lattice girders made from circular hollow sections)

Type of connection		Chords	Verticals	Diagonals
Gap	*K* type	1.5	1.0	1.3
Connections	*N* type	1.5	1.8	1.4
Overlap	*K* type	1.5	1.0	1.2
Connections	*N* type	1.5	1.65	1.25

Table 17.4 Multiplying factors for calculated stress range as per IS: 800 for lattice girders made from rectangular hollow sections

Type of joint		Chords	Verticals	Diagonals
Gap	*K* type	1.5	1.0	1.5
Connections	*N* type	1.5	2.2	1.6
Overlap	*K* type	1.5	1.0	1.3
Connections	*N* type	1.5	2.0	1.4

17.6 General Guidelines for Fatigue-resistant Design

From experience, it is seen that most of the fatigue failures are caused by improper detailing rather than inadequate design. Indeed, weld deflects and poor weld details are the major contributors to fatigue failures (Radaj 1990). Hence the following precautions should be taken when designing and detailing structures (especially with welded joints) subjected to fatigue loads (McGuire 1968; Gurney 1979; Lu & Mäkeläinen, 2003; Maddox 2005).

1. Multiple load paths and/or structural redundancy should be provided in structures to avoid overall collapse of the structure due to the failure of one element under fatigue.
2. Abrupt change in the cross section or stiffness of members should be avoided.
3. Eccentricities should be avoided or they should be reduced to the minimum (Blodgett 1966).
4. Wherever practicable, joints and welds should be restricted to locations of low stress such as at points of contraflexure or near the neutral axis.
5. Details that produce severe stress concentrations or poor stress distribution should be avoided. For example, the fatigue strength of a complete penetration grove welded butt splice in a tension member increases if the weld is ground

flush with the surface of the connecting parts. This eliminates the stress concentration that arises in the as-welded condition. Grinding, however, should not be specified unless essential, since it adds to the final cost (Blodgett 1966). Figure 17.20(a) illustrates a transverse joint in which the weld is elongated in the direction of the load to produce a more uniform transfer of stress than in a conventional weld of Fig. 17.20(b). However, it has to be noted that the code does not permit an increase in the allowable unit stress of such a weld.

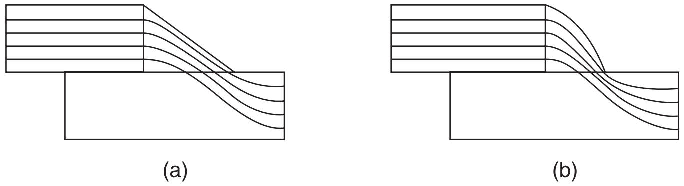


Fig. 17.20 Stress concentration at the toe of a fillet weld

6. Re-entrant, notch-like corners should be avoided; wherever possible gradual change in sections should be provided to produce a smooth ‘stress flow’.
7. Detail that may introduce high, localized constraints should be avoided.
8. Instead of intermittent welds, continuous welds should be used. Intersection of welds should be avoided.
9. Wherever possible, butt or single- and double-bevel butt welds should be used instead of fillet welds. When fillet welds are selected, double-sided fillet welds should be preferred over single-sided fillet welds. Similarly, deep penetration fillet welds should be preferred over normal fillet welds.
10. The size of fillet welds (carrying longitudinal shear) adopted at the site should not be larger than the designed size.
11. Warping and residual stress build-up in butt welds should be minimized by proper edge preparation.
12. Welded joints may be preheated or post-heated to relieve the residual stresses. However stress relieving the weld has been found to have no appreciable effect upon fatigue strength (Bennantine et al 1990).
13. Attachments on parts subjected to severe fatigue loading should be avoided (Blodgett 1966). If unavoidable, they should be welded with a profile that merges smoothly with the profile of the parent metal.
14. Suitable inspection should be provided during the fabrication and erection of structures, especially during welding and tightening of high-strength friction grip bolts.
15. When fatigue cracks are discovered, steps to prevent their propagation should be taken immediately. The repair measures undertaken should not result in a more severe fatigue condition (Kulak & Gilmor 1998).
16. At locations where heavy fatigue loading is expected, such as machine bases subject to dynamic loading or the end connections of large flare booms subjected to wind oscillations, HSFG waisted bolts should be employed. These bolts resist direct tension by means of the pre-stress in the bolts.

17. Corrosion fatigue (especially in offshore structures) may be minimized by particular care in the choice of the material, design, fabrication technique, and the use of cathodic protection. Due attention must be paid to the correct range of potential for cathodic protection. Too low a potential leaves the material susceptible to corrosion fatigue. A too high potential allows hydrogen to enter, leading to the risk of hydrogen embrittlement, if the yield stress is sufficiently high (Clarke & Coverman 1987).

17.6.1 Fatigue Cracking from Out–of–plane Effects

In many instances, it was found that fatigue crack growth resulted from the imposition of relatively small, out-of-plane deformations (Fisher 1978; Kulak & Gilmor 1998). Such deformations can be eliminated by proper detailing. For example, normally transverse stiffeners in plate girders are welded to the web of the plate girders, leaving a small gap at the bottom as shown in Fig. 17.21. It is done mainly to avoid the effect of stress concentration which will increase the fatigue or brittle fracture possibilities. It is also permitted in structural codes. When lateral movement of the top flange relative to the bottom flange takes place, large strains are produced in the gap region due to the significant change in the stiffness between the stiffened and unstiffened (gap) regions of the web. Due to these large strains, only few cycles are required to propagate a crack (Fisher 1978; Fisher & Mertz 1985). Figure 17.21 shows such a crack emanating from the weld toe at the bottom of the stiffener. This crack may also extend across the toe of the fillet weld at the underside of the stiffener and for some distance into the web (Kulak & Gilmor 1998). However, it will not cause any problem, since it is parallel to the direction of the main stress field of the girder in service. However, if the crack is not arrested, it may grow and turn upwards or downwards in the web—which may be an unfavourable orientation with respect to service-load stresses.

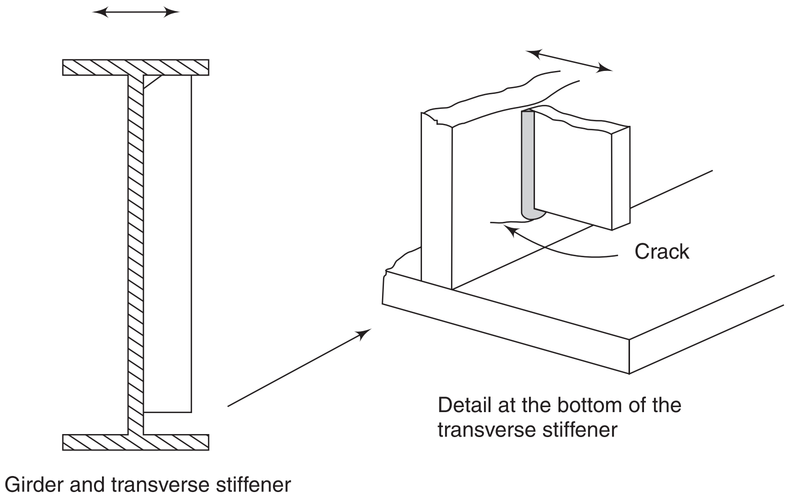


Fig. 17.21 Fatigue cracking due to out-of-plane moment (Kulak & Grondin 2002)

Such flange movements and cracking are possible due to transverse forces in skew bridges. Similar out-of-plane movements have also been found in bridges as a consequence of shipping and handling of such girders. The detail shown in Fig. 17.21 has been identified as a source of many fatigue crack locations in the past (Fisher 1978; Keating 1994). One way of dealing with the crack is to drill a hole of 12 mm diameter in the web of the girder at the end of the crack as shown in Fig. 17.22. It is important to locate and drill the hole at the extremity of the crack tip, such that the hole intercepts the crack front. If necessary, the hole should be ground lightly to remove scratches and burrs. The hole should be left open and monitored for any further crack growth. Sometimes, a high-strength bolt is placed in the hole and pre-tensioned—which will introduce high compressive stresses in the web, thus effectively increasing the fatigue life of this region (Kulak & Gilmor 1998). However, if the original fatigue crack continues to grow, it will be known only after it appears beyond the bolt head or washer. Hence it is better to leave the drilled hole open for easy monitoring.

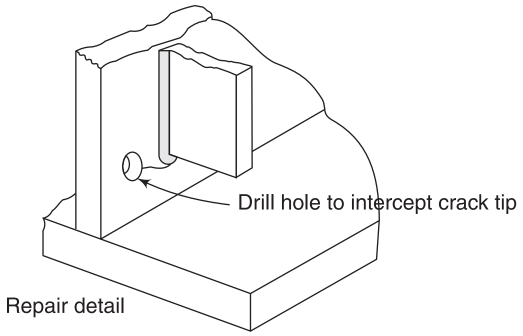


Fig. 17.22 Repair of a fatigue crack (Kulak & Grondin 2002)

Similar out-of-plane movements may occur in a floor beam attached to a connection plate that is welded to the web of the girder as shown in Fig. 17.23. In such a situation, the floor beam tends to rotate under the traffic load. This results in a deformation within the gap at the top of the connection plate (Fisher 1978; Fisher & Mertz 1985). This out-of-plane movement will result in fatigue cracks at the weld at the top of the connection plate or at the web-to-flange fillet weld of the

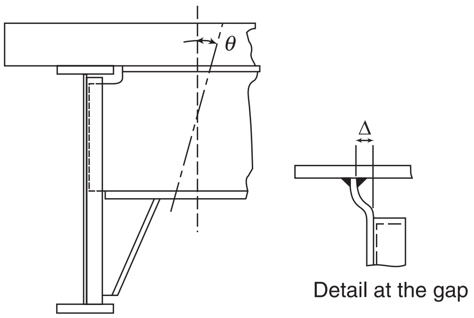


Fig. 17.23 Floor beam-to-girder connection (Kulak & Grondin 2002)

girder, or both, under relatively few cycles of load. Calculations show that a very small displacement, of the order of 0.003 mm, could significantly reduce the fatigue life of this detail (Kulak & Gilmor 1998).

Other sources of out-of-plane fatigue cracking include the following.

Restraint at simple end conditions As discussed in Chapter 2, many simple connections designed based on the assumption that they will not transmit any moment, in reality, carry some moment. Hence the connecting elements may deform under the moment. Thus, end rotation at web framing angles causes the angles to deform and results in additional loads in the connecting bolts and/or rivets, which are not considered in the design. Due to cyclic loading, fatigue cracks may develop in the angles and also in the connecting bolts or rivets.

Coped beams To facilitate easy connection, the ends of beams are often cut back (called coped) as shown in Fig. 17.24. In beams such 'coped' ends (flame cutting the top or bottom flange or both flanges), due to the small radius and unevenness of cutting, are susceptible to fatigue cracking. Normally, coped ends of the beams are locations of zero moments. However, due to the restraint offered by the connection, fatigue cracking may result. Hence, as far as possible, coped ends should be avoided. More information on the design of beams with coped ends may be found in Cheng (1993).

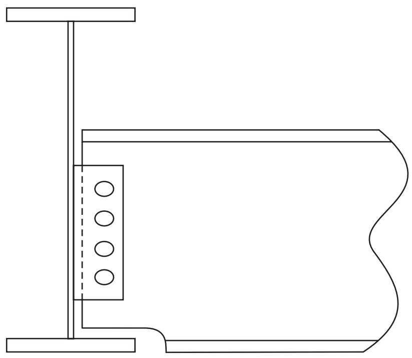


Fig. 17.24 Bottom flange of floor beam coped at connection to girder (Kulak & Grondin 2002)

Bracing connections Lateral bracing attached to horizontal connection plates welded to girder webs also produces out-of-plane flexing of the web and results in fatigue cracking of the connection welds.

17.7 Fatigue Assessment Under Combined Stresses

When both normal and shear stresses are concurrently present in considerable magnitudes at a given location during a given loading event, the principal stresses should be calculated and the stress range corresponding to those stresses should be used in the fatigue life evaluation. (Normal stress may be defined as the stress perpendicular to the direction of the potential fatigue crack.)

When normal and shear stresses are present at the same location but do not occur simultaneously under a given loading event, Kulak and Gilmor (1998) suggest that the individual components of damage can be added using Miner's summation as below:

$$[f_e/f_{fd}]^3 + [\tau_e/\tau_{fd}]^5 \leq 1 \tag{17.24}$$

where f and τ refer to normal stresses and shear stresses, respectively. The subscript 'e' denotes a calculated equivalent stress range [see Eqns (17.21)–(17.23)] and the subscript 'fd' refers to the design fatigue strength for the detail.

17.8 Fatigue Analysis of Building Frames

A fatigue analysis is not presently required for steel structures subjected to wind or earthquakes. Seismic design, as currently prescribed in IS: 800 and other codes, refers mainly to the provision of stiffness, strength, and ductility. The latter is required due to the fact that structural elements are allowed to yield during strong earthquakes. However, such a design methodology inevitably accepts structural damage due to the development of inelastic action. In spite of the recognition of this fact, the fatigue limit state is not introduced in the codes. One reason, of a rather practical nature, is that from the past experience steel structures were not considered vulnerable to earthquake. Another reason is that despite the similarities to the usual fatigue mechanisms, i.e., formation and growth of fatigue cracks, there exist differences to the fatigue due to the usual dynamic loading. That is, fatigue due to earthquake loading is not due to a large number of applied cycles of rather low nominal stress, but due to a small number of applied inelastic deformation cycles, i.e., it is a case of *low-cycle fatigue*. This difference calls for a different assessment procedure of the phenomenon (unlike for high-cycle fatigue), on which a generally accepted agreement is yet to be found.

However, after the Northridge, USA (1994) and Kobe, Japan (1995) earthquakes, it was found that in many cases brittle fractures are initiated within the connections at low levels of plastic demands (and in some cases, especially Northridge) while the structure remained elastic. Extensive analytical and experimental investigations undertaken after these earthquakes revealed that the poor fatigue resistance due to the low material toughness, poor weld execution, high stress concentration of details, restraint conditions, high strain rates, low temperature, etc., were the main causes of failure. Failures were found in regions where high inelastic action develops, such as in the region of beam-to-column-joints. They include rigid welded connections, rigid and semi-rigid bolted end-plate connections, top and seat angle connections, and compact or slender web panels. Structural members outside joint regions have also been found to be prone to fatigue failure when subjected to cycles of inelastic deformations.

Hence a fatigue assessment procedure similar to high-cycle fatigue curves of Figs 17.13 and 17.15, but substituting deformation ranges instead of the stress ranges, has been proposed by Vayas et al. (2003). The relevant fatigue expression in this case is written as

$$\log N = \log C - m \log \Delta_p \tag{17.25}$$

where Δ_p is the fatigue deformability (plastic rotation), N is the number of rotation range cycles, m is the slope constant of a fatigue curve, and log C is a constant.

A transformation of elastic and inelastic deformation into equivalent stresses has also been proposed by Ballio and Castiglioni (1995). However, the fact that the resulting stresses may be well above the material tensile strength might be confusing for the application.

In the absence of more specific information, the damage assessment for variable ranges of plastic rotation may be performed as for high-cycle fatigue in accordance with the linear Palmgren–Miner cumulative law [Eqn (17.17)]. The value of m in Eqn (17.25) ranges between 1.3 and 3.4 for welded connections and top and seat angle connections. For 'log C' values and other details of low-cycle fatigue assessment, the reader may consult Vayas et al. (2003), Ballio and Castiglioni (1995), Mazzolani (2000), Righiniotis et al. (2000) and Vayas et al. (1999).

Table 17.5 summarizes the various possible formulations of the fatigue analysis as well as examples of relevant applications. Mohammadi (2004) gives details of non-destructive testing methods applied to the fatigue reliability assessment of structures.

Table 17.5 Formulations of the fatigue rules

Structural response	Elastic → inelastic		
No. of cycles to failure	$\sim 10^4$–10^8	$\sim 10^2$–10^4	$\sim 10^0$–10^2
Fatigue curves for ranges of	Generalized forces	Generalized total deformations	Generalized inelastic deformations
Usual field applications	Bridges, crane girders chimneys, masts, etc.	Slender plate girders	Buildings under seismic loading

Source: Vayas et al. (2003)

Examples

Example 17.1 *An overhead crane in a factory has a gantry girder with a simply supported span of 6.0 m. The section used for the girder is ISMB 400. The capacity of the crane is 150 kN and the gantry girder receives a maximum of 80% of the total load as the reactive force. It is assumed that this force is applied to the girder as a single concentrated load. The crane operates 200 days/year and 8 hr/day. The crane makes a maximum of three trips per hour at this load level. The design life of the building is 50 years. Is the fatigue life of this gantry girder satisfactory?*

Solution

Number of stress cycles (equals number of load cycles in this case)

$$N = (3 \text{ cycles/hr})(8 \text{ hr/day})(200 \text{ days/year})(50 \text{ years}) = 240{,}000 \text{ cycles}$$

Detail category and fatigue strength

The provided beam corresponds to detail category 118 as per Table 13.3(1) of the code. From Fig. 17.13, read the category 118 line at N = 240,000 cycles to find that uncorrected fatigue strength is 330 MPa. The fatigue strength of this detail can also be calculated using Eqn (17.13) as

$$f_f = 118 \times \sqrt[3]{(5 \times 10^6 / 240{,}000)} = 324.6 \text{ MPa}$$

Assuming fail–safe and poor accessibility for detail, $\gamma_{\text{mft}} = 1.15$

Design fatigue strength = 324.6/1.15 = 282.26 MPa

Calculation of actual stress range

$$f_{\min} = 0$$

$$M = PL/4 = (150 \times 10^3 \times 0.8)(6000)/4 = 180 \times 10^6 \text{ N mm}$$

$$f_{\max} = M/Z = 180 \times 10^6/(1022.9 \times 10^3) = 176 \text{ MPa} < f_y = 250 \text{ MPa}$$

Thus $\quad f = 176 - 0 = 176$ MPa.

Since the actual stress range (176 MPa) is less than the design fatigue strength (282.26 MPa) for this detail, the size of girder is safe.

Shear force as support = $150 \times 10^3 \times 0.8/2 = 60{,}000$ N

Shear stress = SF/(dt_w) = 60,000/(400 × 8.9)

= 16.85 MPa < (27/1.15 = 23.48 MPa)

Hence fatigue assessment is not required at the support. However, let us find out the fatigue strength in shear of the detail using Eqn (17.15).

$$\tau_f = \tau_{\text{fn}} \sqrt[5]{5 \times 10^6 / N_{\text{sc}}}$$

$$= 118 \sqrt[5]{5 \times 10^6 / 240{,}000}$$

$$= 216.58 \text{ MPa}$$

Design fatigue strength in shear = 216.58/1.15 = 188.3 ≫ 16.85 MPa

However, the following points have to be borne in mind.

1. The number of stress cycles will not always be equal to the number of load cycles. For example, in a continuous beam or when a multiple-axle vehicle transverses a member, more than one stress cycle could occur.
2. Since the stress due to the dead load is always present, the change in stress in the member could be taken equal to the change in stress produced by the moving (live) load.
3. Of course, we can also compare the number of cycles permitted at the actual stress range of 176 MPa.

From Table 17.1, log C for category 118 and $N < 5 \times 10^5 = 12.901$

From Eqn (17.6),

$$\log N = 12.901 - 3 \log 176$$

Thus $\quad N$ = 1,460,366 cycles

Since the actual number of cycles is only 240,000, this also shows that the size of the girder is sufficient.

Example 17.2 *After erecting the gantry girder of Example 17.1, it has been decided for the crane to accommodate two trips per hour at a load level of 180 kN, in addition to the existing conditions. Will the fatigue life of this gantry girder be still satisfactory?*

Solution

Number of cycles at the old load level, $n_1 = 240{,}000$

Number of cycles to failure at the old load level, $N_1 = 1460{,}366$

Number of cycles at the new load level,

$$N_2 = (2 \text{ cycle/hr})(8 \text{ hr/day})(200 \text{ days/year})(50 \text{ years})$$
$$= 160{,}000 \text{ cycles} < 5 \times 10^6 \text{ cycles}$$

Number of cycles to failure at the new load level

$$M = \text{PL}/4 = (180 \times 10^3 \times 0.8)(6000/4) = 216 \times 10^6 \text{ N mm}$$

$$f_{\max} = 216 \times 10^6/(1022.9 \times 10^3) = 211.2 \text{ MPa} < f_y = 250 \text{ MPa}$$

Using Fig. 17.13 for category 118 or using Eqn (17.6),

$$\text{Log } N_2 = 12.901 - 3 \log 211.2,\ N_2 = 845,\ 119 \text{ cycles}$$

Checking Eqn (17.18),

$$\Sigma n_i/N_i \le 1$$

we get

$$240{,}000/1460{,}366 + 160{,}000/845{,}119 = 0.35$$

Since the total effect of the two different stress ranges is less than 1.0, the gantry girder is safe for the additional loading condition.

Example 17.3 *Use the equivalent stress method to determine the percentage of life that has been expended by the loading applied to the gantry girder of Example 17.2. Note that the Palmgren–Miner cumulative fatigue damage rule [Eqn (17.17)] can be expressed in terms of an equivalent stress range f_e (as per Kulak & Gilmor, 1998) as*

$$f_e = [\Sigma \alpha_i\, f_e^{\,m}]^{1/m} \tag{17.26}$$

where

$$\alpha_i = n_i/\Sigma N_i \tag{17.27}$$

Solution

All the necessary data as available from Example 17.2 are given below:

$$n_1 = 240{,}000,\quad n_2 = 160{,}000$$

$$f_1 = 176 \text{ MPa},\quad f_2 = 211.2 \text{ MPa}$$

$$N = 240{,}000 + 160{,}000 = 400{,}000 \text{ cycles}$$

Using Eqn (17.26)

$$f_e = [(240{,}000/400{,}000)(176)^3 + (160{,}000/400{,}000)(211.2)^3]^{1/3}$$
$$= 191.7 \text{ MPa}$$

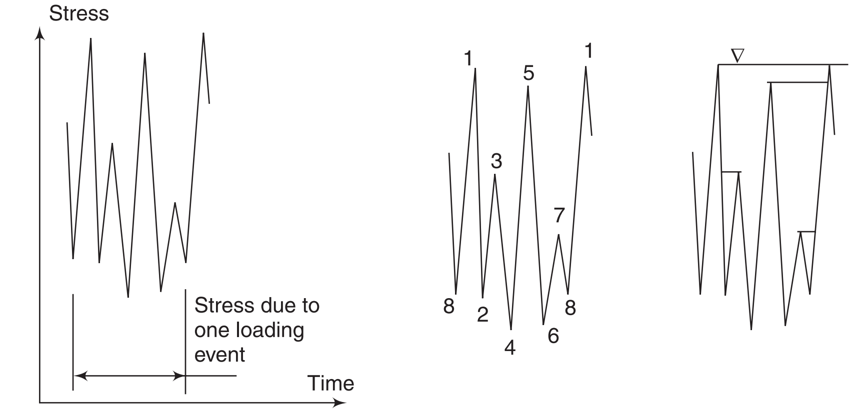
Thus the number of cycles to failure

$$\text{Log } N = 12.901 - 3 \log 191.7 \quad \text{or} \quad N = 1130{,}143 \text{ cycles}$$

Since the actual number of cycles is 400,000, the percentage of life expended is $(400{,}000/1130{,}143) \times 100 = 35\%$

Note that we get the same result using Miner's summation.

Example 17.4 *The numerical values of the peaks and valleys for the stress history are shown in Fig. 17.25.*



(a) An example of stress variation in an element due to one loading event
(b) Peaks and troughs for one loading event
(c) Reservoir analysis

Fig. 17.25 Reservoir analysis of stress histories (Kulak & Grondin 2002)

Peak/Trough no.	Stress (in MPa)
1	96
2	15
3	47
4	10
5	80
6	10
7	35
8	15

Apply the reservoir counting method in order to identify the stress ranges in this stress spectrum and evaluate the effects of 3.0 million loading events of this stress history acting on a welded I-beam that uses longitudinal fillet welds to connect the flanges to the web.

Solution

This detail is category 92 as per Table 26b of the code (see also Fig. 17.14). The reservoir counting method is presented in a tabular form.

Drain from trough no.	Water level at peak no.	Stress range (MPa)
4	1	96–10 = 86
6	5	80–10 = 70
2	3	47–15 = 32
8	7	35–15 = 20

The damage resulting from each of these four stress ranges can now be calculated in a tabular form as on the next page.

Stress range, f_f (Mpa)	Fatigue resistance (cycles)	Damage due to 3.0×10^6 loading cycles
86	6,273, 424[a]	0.478[b]
70	11,633,379	0.258
32	121,772,736	0.025
20	498,781,128	0.006
	Total	**0.767**

[a]Using Eqn (17.6): log N = 12.601 – 3 log 86, N = 6,273,424 cycles
[b]Using Eqn (17.17): $n_i/N = 3.0 \times 10^6/6{,}273{,}424 = 0.478$

Since the damage summation is only 0.767 (less than unity) the detail will be safe to take the stress history shown in Fig. 17.25 without fatigue failure.

Summary

In most practical situations, the type of loading will not be monotonic, i.e., it will not increase steadily until failure occurs. Instead, the applied stresses fluctuate, often in a random way (Agerskov 2000). In some cases, the stress at a given point on the surface varies cyclically from tension to compression and back again. In the presence of cyclic stresses, it is found that premature failure occurs at stress levels much lower than those required for failure under a steadily applied stress. This phenomenon of premature failure under fluctuating stress is known as fatigue and involves processes of slow crack growth.

Components of buildings are not subjected to fatigue, but bridges, gantry girders, slender towers, offshore platforms, and structures supporting large rotating equipment are prone to fatigue. Fatigue failures may be classified as high-cycle and low-cycle failures. Stresses associated with the low-cycle fatigue are generally high and the fatigue life is described in terms of strain range (used in ground vehicle industries). Under high-cycle, low-stress fatigue, the material deforms primarily elastically and the failure life is described in terms of stress range (as in structural applications).

Fatigue crack initiation is mainly due to the effect of the stress concentration or flaw in the material. In welded steel structures, fatigue cracks start to grow from the imperfections or metallurgical discontinuities in the welds. The growth rate of a crack is proportional to the square root of its length and, hence during the initial life, the cracks are very small and hard to detect. Only in the last stages of life do the cracks cause significant loss of cross section which leads to failure. Due to this, it is very difficult to identify the crack growth in the initial stages, unless we use non-destructive testing methods.

The number of cycles of loadings, the stress range, and the type of member detail at the location are the three important parameters that affect the fatigue life of the component. Other factors that influence the fatigue behaviour are the stress concentration, residual stress, plate thickness, imperfections, stress ratio, frequency of cyclic loading, post-weld treatment, temperature, and environment. The presence or absence of non-metallic inclusions in the material also can have a significant effect on the fatigue life. Fatigue strength is always checked under service-load conditions.

Though there are different approaches to fatigue analysis, stress method, using *S-N* curves, is often used for structural applications. *S-N* curves show the relationship between the number of cycles for fracture and the maximum stress range due to the applied cyclic load. The fatigue strength of different welded details varies according to the severity of the stress concentration effect. In the code the most commonly used details are grouped into 15 main classes; also, the *S-N* curves are provided for these 15 main classes. The data provided in these curves have been obtained from a large amount of identical test specimens subjected to defined loading conditions and measuring the number of cycles required for failure. *S-N* curves for shear stresses are also provided. A correction factor to take into effect the influence of the thickness of the plate is also specified.

The *S-N* curves given in most of the codes are for fully reverse load ($R = -1$) and zero mean stress. However, non-zero mean stress can also play an important role in fatigue design. In such cases the modified Goodman diagram is used.

The fatigue life of components under variable-amplitude loading is calculated using the Palmgren–Miner rule, which is a linear damage rule. The two main shortcomings of the linear damage rule (assuming sequence independence and damage accumulation independence) can be overcome by non-linear damage rules. However, for structural applications, the Palmgren–Miner rule is found to give satisfactory results. When the linear Palmgren–Miner rule is used, the variable-amplitude loading needs to be transformed into a series of constant-amplitude loading. Out of the several methods available for cycle counting, reservoir counting and rainflow counting methods are explained with examples. Discussions on the fatigue assessment of hollow sections and guidelines for the fatigue-resistant design and detailing are provided. Fatigue cracking from out-of-plane effects and the methods to mitigate them are also discussed. An equation for the fatigue assessment under combined stresses is also given. The fatigue assessment procedure for steel structures subjected to earthquake, based on deformation range (instead of the stress range of high-cycle fatigue curves), is also given based on research studies.

Exercises

1. A crane girder of section ISMB 500 having a simply supported span of 8 m has to be checked for fatigue. The capacity of the crane is 300 kN and the gantry girder receives 75% of the total load as the reactive force. Assume that this force is applied as a single concentrated load. The crane operates 250 days/year and 6 hr/day. The crane makes a maximum of three trips per hour at this load level. Assuming the design life of the building to be 60 years, determine the fatigue assessment of this gantry girder.

2. After erecting the gantry girder in Exercise 1, it was decided to accommodate two more trips per hour at the same load level. Will the fatigue life of the gantry girder be still satisfactory?
3. Use the equivalent stress method to determine the percentage of life that has been expended by the loading applied to the gantry girder of Exercise 2.
4. The numerical values of peaks and valleys for the stress history of a gantry girder are shown below (see also Fig. 17.25)

Peak/trough no	Stress (in MPa)
1	93
2	18
3	55
4	8
5	85
6	10
7	37
8	18

Apply the reservoir counting method in order to identify the stress ranges in this stress spectrum and evaluate the effect of 3.5×10^6 loading events of this stress history acting on a plate girder with multiple cover plates (category 37 of code).

Review Questions

1. What is meant by fatigue and fatigue damage?
2. Why fatigue is not considered in the design of members in building frames?
3. Which structures are prone to fatigue failure?
4. Name the four stages of fatigue failure.
5. Explain the difference between high-cycle and low-cycle fatigue failures.
6. What are the two important factors that result in the initiation of fatigue cracks?
7. What flaws are there in rolled shapes?
8. How flaws may be introduced due to fabrication and erection?
9. Why most of the fatigue cracks may start from welds in a welded structure?
10. Name some of the imperfections that are found in a weld.
11. How are imperfections caused in a weld?
12. Name the three important factors that affect fatigue life.
13. What are the other factors that may influence the fatigue behaviour?
14. Write short notes on the following factors which influence fatigue behaviour:
    (a) Stress concentration
    (b) Residual stress
    (c) Plate thickness
    (d) Imperfections
    (e) Stress ratio
    (f) Frequency of cyclic loading
    (g) Post-weld treatment

15. What are the three different approaches to fatigue analysis?
16. Define stress range, mean stress, stress amplitude, and stress ratio.
17. What is an *S-N* curve?
18. Write the relationship between stress range and fatigue life as given in the code.
19. Define fatigue limit and fatigue strength?
20. What is the fatigue limit for normal steel members?
21. What are the factors that affect the *S-N* curve?
22. What are constant-amplitude fatigue limit and cut-off limit?
23. What is the slope (m value) for fatigue strength curves used for shear stresses?
24. State the conditions which when satisfied eliminate fatigue assessment.
25. What are the partial safety factors for fatigue strength?
26. State the equation which takes into account the influence of the thickness of plate.
27. State the fatigue strength equations of a standard detail as given in the code for normal and shear stresses.
28. How is the modified Goodman diagram constructed?
29. State the Palmgren–Miner rule.
30. What are the different methods used for cycle counting?
31. Illustrate the procedure of cycle counting using the rainflow method.
32. List some of the guidelines to be followed while designing and detailing welded joints for fatigue loads?
33. How can we arrest and repair a fatigue crack emanating from out-of-plane effects?
34. Why are coped ends of beams susceptible to fatigue cracking?
35. State the equations (Miner's summation) to take into account combined stresses?