



The Welding Engineer's Guide to Fracture and Fatigue

Philippa Moore and Geoff Booth

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*Philippa would like to dedicate this book to her family, especially her children
Rosie and Jonas.*

Geoff would like to dedicate this book to his family.

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Preface

Welding engineering is a discipline that many engineers find themselves moving into only once they begin to specialise within their career, because it is not a common area to study at college or university. Some begin as welders but later wish to acquire the knowledge to become welding supervisors, welding inspectors or welding engineers. Others are tempted to consider a career change by the potential opportunities offered by welding engineering, or simply by curiosity of this broad and industrially significant subject. Whatever the reason that has caused you to begin to read this book, we hope to inform and enlighten you into some of the aspects of fracture and fatigue that must be understood in order to become a welding engineer.

No prior knowledge of the subjects of fracture and fatigue is assumed and the concepts are explained from first principles. Unless absolutely necessary, we avoid using mathematical formulae to explain concepts as much as possible. We hope that to learn more about fracture or fatigue as subjects in their own rights you use the lists of further reading given at the end of each chapter. Both fracture and fatigue are fields where there are many academic textbooks and large amounts of ongoing research. However, this book is for welding engineers, and we have selected those parts of these topics which relate specifically to welds, and are valuable for a welding engineer, particularly in their discussions with fatigue designers or fracture mechanics experts. Part One covers the basic principles and includes descriptions of typical welding defects and how these defects behave in structures under static and cyclic loading, potentially leading to premature failure. Part Two then provides guidance on how to avoid such failures, through selection of appropriate materials, welding procedures and inspection regimes for the intended service conditions. The technical content is relevant to all welded engineering structures, including pipelines, fixed and floating offshore platforms, bridges, buildings, ships, pressure vessels and off highway vehicles. Each chapter may be read independently, and gives references to other chapters when further relevant information is found in other parts of the book.

This book is written with the intention of accompanying the Design and Construction modules of the IIW Welding Diploma at all levels, to provide all the information on fracture, fatigue, and related topics in the syllabus. The authors have taught the Design and Construction modules at TWI in Cambridge for several years and decided to write this book to give extra support to those students, and to others worldwide. We have included the topics that students at the higher levels of Technologist and Engineer need, although Specialist level students would not be expected to know the material in Chapters 5, 11 and 12. The descriptions of testing and inspection methods in Chapters 9 and 10, though not strictly necessary for the Design and Construction module, are required in other modules of the Diploma, and have been included to make the whole book more complete in itself. We are grateful

to TWI Ltd for their support in writing this book, and for permission to use many of its corporate images to illustrate the chapters. We also thank many of our friends who have helped along the way with sourcing the images and reviewing chapters.

We hope you find our book to be a helpful resource in your study of welding engineering.

Philippa Moore and Geoff Booth
2014

Designing engineered structures

1

1.1 Introduction

Engineered structures are used frequently on a daily basis. However, the accumulated science, technology, and experience that are required to ensure their safe operation are largely unrecognised and unappreciated.

For our purposes, we define engineered structures as those structures intended to withstand loads or contain pressure. To highlight some common examples, in a typical day we may:

- drive a car: the car carries a load (us and our belongings), the internal combustion engine contains pressure;
- ride a bicycle: the bicycle carries us;
- drive or cycle over a bridge: the bridge supports the weight of the car or bicycle. It also supports its own weight and withstands any environmental forces such as loads caused by wind or river current;
- heat our home and cook using gas: the gas is transported long distances through pressurised pipelines;
- heat our home and cook using electricity: a large proportion of our electricity is generated by boiling water and using steam at pressure to drive turbines; and
- work, live, shop or go to sports events in buildings: these buildings may be very large or tall and have to support their own weight whilst also withstanding environmental forces.

1.2 The first engineered structures

It is natural, perhaps, to think that engineered structures have become commonplace only in the last 200 years or so. In reality, however, structures have a much longer history. The Great Pyramid of Giza, Egypt was built around 4500 years ago (Figure 1.1). It is believed to be a monumental tomb constructed from several million stone blocks, each weighing about 2 tonnes, and it is more than 130m tall. The pyramid contains passageways and chambers and was built with a high degree of geometric precision. Details of the method of construction are now lost, and there is no evidence remaining of how it was designed, if indeed it was designed at all in the modern sense. Nevertheless, the Great Pyramid is an engineered structure: it has supported its own weight, would have fulfilled its design criteria of providing a tomb and



Figure 1.1 The Great Pyramids of Giza Egypt (© Lisa Webber).



Figure 1.2 Stone structures at Stonehenge, UK (© Simon Bird).

treasury but for grave-hunters, and has suffered only minor environmental damage in its lifetime of several millennia.

Stonehenge, a 30m diameter ring of standing stones in England, was also built around 4500 years ago (Figure 1.2). Each stone weighs about 25 tonnes and is about 4m high; many of the upright stones support horizontal lintel stones which weigh roughly 10t. The construction techniques employed are unknown, and the purpose of Stonehenge is now largely a matter of conjecture, although the alignment of the stones is believed to be astronomically significant. The concept of a design life was probably not considered in any formal way, but it is a very substantial achievement

to have built such a structure that is still standing long after its purpose has been forgotten.

Both the Great Pyramid and Stonehenge have successfully withstood their self-weight and environmental forces for a very long time, and are early examples of engineered structures. The people who built them must be considered pioneering engineers and engineering therefore has a very long tradition.

1.3 Successful structures

To evaluate whether a structure has performed successfully, we need to develop our earlier very broad definition of an engineered structure. In reality, engineered structures must withstand loads or contain pressure AND operate safely to meet specified performance targets.

In the cases of the Great Pyramid and Stonehenge, it is unlikely that the specified performance targets were ever even considered; it was probably simply assumed that they would last forever. There are many other examples of historical structures that were built with limited, if any, consideration of lifetime. These include the immense stone cathedrals of the Middle Ages and the great Elizabethan mansions.

In modern design, very careful consideration is given to what is meant by a performance target. For a bridge, this may be the ability to carry a certain number of vehicles of a particular weight for a specified number of years. A fixed offshore platform may have to withstand waves, currents, and storms for a certain number of years and economically produce oil and gas in that time, Figure 1.3. A gas or oil pipeline is required to transport potentially hot and/or corrosive fluids under pressure without failure for its entire lifetime (Figure 1.4).



Figure 1.3 Typical design of a fixed offshore structure, such as the base for an offshore oil or gas platform in the North Sea (© TWI).



Figure 1.4 Girth weld in a gas transmission pipeline in the UK before being buried (© TWI).

1.4 Materials and fabrication methods

A wide variety of materials can be used to build engineered structures. The Pyramids, Stonehenge and the medieval cathedrals were built using stone. Timber has also been used extensively, but its lifetime tends to be measured in centuries rather than millennia.

The vast majority of engineered structures today are made from metals, with steel being by far the most common alloy used. Aluminium alloys (and in some instances magnesium alloys) are used where weight has to be minimized, such as in aircraft. Titanium alloys are used where a high strength-to-weight ratio is required, and Ti alloys also have good high-temperature strength, making them ideal for use in aircraft engines. Composite materials are also used in engineered structures, for example fibre-reinforced plastics for aircraft airframes and steel-reinforced concrete in bridges and buildings.

This book is for welding engineers and is predominantly concerned with steel structures. Where relevant, comparisons are made between the behaviour of steel and that of aluminium alloys. Other engineering materials are beyond the intended scope of this book.



Figure 1.5 An example of a typical riveted construction railway bridge, crossing the river in Cambridge, UK (© TWI).

With very few exceptions it is not possible to make an engineered structure from a single monolithic piece of material. Elements of a structure must therefore be joined together in order for it to fulfil a useful function. Until the late 19th century the main methods of joining steel plates and members together were riveting and bolting (Figure 1.5). Rivets are a type of mechanical fastener and many bridges, ships, railway engines and boilers were historically joined using this method. Bolting is also a well-established type of mechanical fastener. However, both these joining techniques are relatively slow, inflexible, and labour intensive.

Welding is a joining technique that forms a metallurgical bond between parts by the application of heat or pressure or both. It became an important fabrication process early in the 20th century. Welding is probably the most adaptable and efficient of all joining processes and is almost always the joining process of choice when fabricating steel structures. It is not perfect, however, and the advantages and problems with welding are discussed further in Chapter 3. Given the dominance of welding in modern structural fabrication, this book restricts itself, therefore, to the performance of welded joints.

1.5 Industrialisation: benefits and consequences

Industrialisation, which started in the late 18th century, brought many economic and societal benefits. Improved methods of making iron (and later steel) enabled large quantities to be produced in new forms at greatly reduced cost. Iron, therefore, became the principal construction material and it became possible to build elegant



Figure 1.6 The world's first cast iron bridge at Ironbridge, Shropshire, UK, built in 1781 (© TWI).

and innovative structures such as the bridge over the river Severn at Ironbridge, England (Figure 1.6). This bridge was constructed in 1781 and is still functioning as a bridge more than two centuries later.

In addition to the improved availability of iron, new sources of power were being developed at this time. Initially, water falling under gravity was used to power machinery, but steam-based sources rapidly became very extensively used because of their greater flexibility and ability to provide power on demand. Steam engines were used for pumping water out of rivers, driving rotary mine hoists and as stationary engines for pulling trucks along railway tracks. Later, steam engines were put on wheels and the steam power was used to move the engine itself, and to pull trucks; the railway industry followed.

From an engineering viewpoint, the main consequence of industrialisation was that much greater demands were made of the new iron and steel structures. For example, bridges had to carry heavier loads: firstly, they spanned larger gaps and hence had greater self-weight and secondly, the vehicles and carriages crossing them were much heavier. In addition, for the first time structures had to withstand many repeated applications of load, instead of either a single or very small number of applications. Examples include railway axles and mine hoists where each rotation gave rise to an application of load and there could be thousands of repetitions each day.

The steam engine itself also presented new challenges. The boiler was required to contain steam at high pressures (typically, 10 atmospheres) and high temperatures (around 200°C) for long periods of time. Additionally, the steam was potentially corrosive and both the temperature and pressure varied over a typical duty cycle.

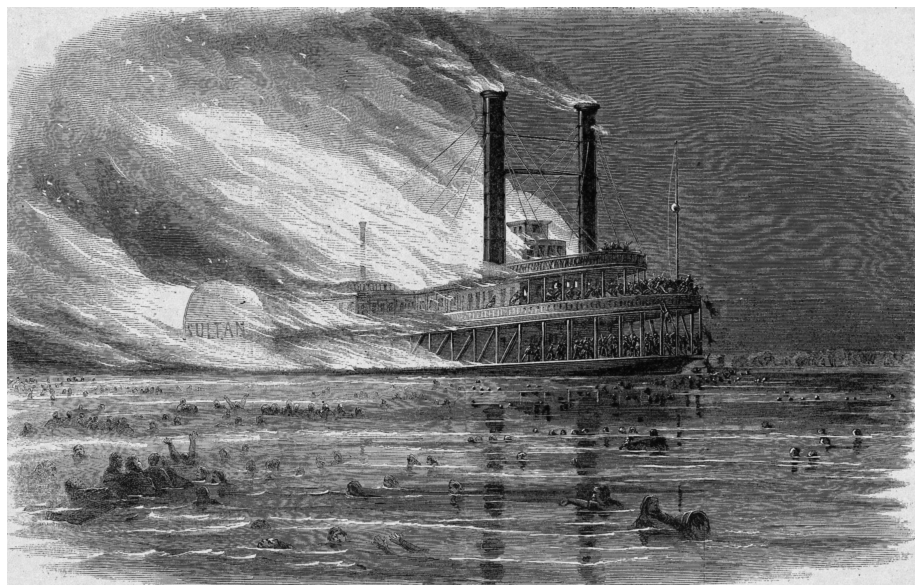


Figure 1.7 The illustration of the SS *Sultana* disaster as reported in the newspaper *Harper's Weekly* in the USA in 1865.

Given the entrepreneurial spirit at the time of industrialisation, it is perhaps not surprising that knowledge of the behaviour of iron and steel when subjected to many applications of load at high temperature was not given a great deal of attention. As industrialisation spread, steam boiler explosions occurred, and by the mid-19th century there were typically several catastrophic failures per year worldwide, each one resulting in deaths and injuries.

One example of a catastrophic failure was the SS *Sultana*, a steam-powered paddle ship operating on the Mississippi river in the USA. In 1865, the vessel was hopelessly overcrowded carrying Union soldiers home after their release from Confederate Prisoner of War camps at the end of the American Civil War. Shortly after leaving port, the boiler exploded and the ship sank in flames (Figure 1.7). The death toll is not known, but it is believed around 1600 people lost their lives in the accident.

A boiler explosion that had far-reaching consequences occurred at the Grover Shoe Factory in Massachusetts, USA, in 1905. The factory was heated by steam and the boiler exploded causing catastrophic damage. Some 58 people were killed and 150 injured, including many women and children. As a result of this tragic loss of life, the public mood changed from fatalistic acceptance of disasters to demanding that no future explosions should be tolerated. This, in turn, led firstly to the development of best practice guidelines and then to the drafting of legal codes for the construction and operation of boilers. The first 'boiler laws' were issued in 1908, and the Boiler and Pressure Vessel Code was published by the American Society of Mechanical

Engineers (ASME) in 1915. Over the last century the code has been expanded and improved to take account of technical developments and research and it is now used worldwide.

The construction of pressure equipment continues to be subject to heavy regulation, with the Pressure Equipment Directive becoming law across the European Union since 1999, giving rules about the construction and safety of pressure equipment.

Other engineering structures are subject to their own construction codes and standards, but outside formal international legislation. More information about design approaches is given in Chapter 4.

1.6 Conclusions

Engineering structures are required to withstand loads or contain pressure, whilst satisfying specified performance targets. Although many materials and joining processes are available to fabricate engineering structures, this book is concerned with steel structures fabricated by welding.

The number of engineering structures and their applications has increased rapidly since industrialisation started in the 18th century. There have, however, been tragedies associated with catastrophic structural failures. In response to this, design philosophies have been developed and formal codes introduced to establish the level of safety that our society demands. It is the intention of this book to provide an introduction to the design and assessment principles currently adopted in order to prevent failure by two of the most important potential failure modes; fracture and fatigue.

1.7 Sources of further information and advice

- American Society of Mechanical Engineers (2013) *Boiler and pressure vessel code*, ASME.
- Arya, C. (2009) *Design of structural elements: concrete, steelwork, masonry and timber designs to British Standards and Eurocodes, Third Edition*, CRC Press; ISBN-10: 0415467209, ISBN-13: 978-0415467209.
- Gordon, J. E. (2003) *Structures: or why things don't fall down, 2nd edition*, DaCapo Press. ISBN-10: 0306812835, ISBN-13: 978-0306812835.
- McEwen, A. (2009) *Historic steam boiler explosions*, Sledgehammer Engineering Press Limited ISBN-10: 0953272524, ISBN-13: 978-0953272525.

2.1 Introduction

In the previous chapter we defined engineered structures as those that are intended to withstand loads or contain pressure in order to satisfy specified performance targets. We identified some of these structures that are an integral part of everyday life, and summarised the fatal consequences that can arise when engineered structures fail catastrophically.

In this chapter we describe the sources and types of the loads that have to be withstood, and the general design principles in common use, including ultimate and performance limit states.

2.2 Sources of loading

There are many different sources of loading on structures and it is useful to categorise them based on the degree to which they vary with time.

2.2.1 *Permanent loads*

Loads that to all intents and purposes do not vary with time are called permanent loads. Examples include the self-weight of the component, hydrostatic pressure at the base of a fixed offshore structure (such as the wind turbines shown in Figure 2.1), and the internal pressure of a gas or oil pipeline.

2.2.2 *Variable loads*

Loads that do vary significantly with time are called variable loads. These loads may arise from the functions of the structure itself, for example, vehicles of differing sizes crossing a bridge, rotations of wind turbine blades, or a crane raising and lowering pallets. Variable loads also include those necessarily associated with the position of the structure, such as wind and wave action on ships and offshore structures, wind blowing across bridges, and snow loading on the roof of buildings (Figure 2.2).

Loads resulting from an imposed deformation such as differential thermal expansion, may be considered as either permanent or variable, depending on the circumstances.



Figure 2.1 Offshore wind turbines experience permanent, cyclic and accidental loading (© TWI).



Figure 2.2 Snow loading on the roofs of buildings in the UK (© TWI).

2.2.3 Accidental loads

As the name suggests, accidental loads result from low probability events such as vehicle or ship impact, explosion, acts of terrorism or earthquake. Although often distinct from the intended function of the structure, such loads are included as a fundamental part of the design process. For example, the side impact bars in a car



Figure 2.3 Side impact protection bars in a car frame, (darker diagonal bars) (© TWI).

(Figure 2.3), are included solely to account for accidental loads, which protect passengers in the event of a side-impact collision. Accidental loads often considered in the design of wind turbines (Figure 2.1) include those from ship collisions or bird strikes.

It is the responsibility of the engineer to identify all sources of loading that are relevant, and then ensure that the structure is adequately designed to withstand them. Table 2.1 illustrates some of the loads that need to be considered for some common structures.

2.3 Types of loading

Table 2.1 lists some of the loads that act on typical structures. However, there may be several sources of load that contribute to a total permanent load; similarly there may be a number of sources of variable loads. By considering all the sources of loading we can arrive at the three main types of loading.

2.3.1 *Static loads*

These are essentially the sum of all the permanent loads. To all intents and purposes these loads do not change significantly over the lifetime of the structure; hence the name ‘static’. In some cases, the static load obtained by adding all the permanent loads together, is the maximum load that the structure has to withstand. In other instances, variable loads are imposed on the permanent load, and the maximum load the structure has to withstand is the sum of the permanent loads, plus the maximum

Table 2.1 Examples of permanent, variable and accidental loads

Structure	Permanent loads	Variable loads and transients	Accidental loads
Road bridge	Self-weight	Vehicle crossings Wind Ice/snow	Vehicle impact Earthquake
Building	Self-weight Floor loading	Wind Ice/snow	Earthquake Fire
Gantry crane	Self-weight	Payload Wind Ice/snow	Earthquake Helicopter collision
Ship	Self-weight	Cargo Wind Wave Pressure on hull Ice/snow Engine vibrations	Impact
Offshore structure	Self-weight Deck load	Waves Current Wind Ice/snow	Ship impact
Land pipeline	Internal pressure	Vibration Pressure fluctuations	Earthquake Landslip
Boiler or pressure vessel	Internal pressure	Vibrations Start up Shutdown Temperature change	Impact

variable load. For example, a bridge has to have sufficient static strength to carry its own weight plus the load arising from a combination of carrying traffic and withstanding wind forces.

2.3.2 Cyclic loads

Cyclic loads correspond to the variable loads already described, although accidental loads such as unexpected vibrations can also contribute to cyclic loading. Cyclic loads are expected to vary, usually in a systematic manner, throughout the structure's lifetime. They may arise as a result of the job the structure is expected to perform,



Figure 2.4 The fuel tanker *Whitonia*, which provides fuel for cruise ships in Southampton harbour from the nearby Fawley refinery (© TWI).

e.g. a crane repeatedly lifting weights, or as a result of location, such as a ship exposed to wind and waves. Figure 2.4 shows the fuel tanker ‘Whitonia’, which provides fuel for cruise ships in Southampton harbour from the nearby Fawley refinery; a ship such as this experiences cyclic loading as a result of the filling and emptying cycles during fueling, as well as wind and wave loading from the sea.

2.3.3 Impact loads

Impact loads are applied rapidly and usually correspond to accidental loads. Components designed for impact loads include tools (such as hammers or forging tools), crash barriers by the side of roads, and ballistic protection for armoured vehicles and body armour.

2.4 Loads experienced during construction

Here we are concerned with the behaviour of steel, particularly welded steel joints, when subjected to static and cyclic loads. It is perhaps natural to consider the structure only as a completed construction. Whilst being built, however, the structure may experience significantly different loads. As a simple example, consider a bridge across a gorge being built from both banks simultaneously. As the bridge deck is being built outwards, it acts as a cantilever, but when the two projections meet and are joined together, the deck acts like a beam supported at each end. The loads during construction and operation must be considered separately to assure safety and integrity. This point was emphasised during construction of the Westgate Bridge in Melbourne. This box



Figure 2.5 Westgate Bridge in Melbourne as the construction was nearing completion in 1970 (copyright courtesy of Pat Preston, ex Westgate Bridge crane driver).

girder bridge collapsed during construction in 1970, killing 35 construction workers and welders who were on or underneath the bridge at the time of the collapse (Figures 2.5 and 2.6). Failings in the design to provide sufficient safety factors on the loads during construction (amongst numerous other factors) contributed to the failure.

2.5 Design approach

Modern design approaches are based on the concept of a 'limit state' beyond which the structure can no longer meet its design targets. There are many different 'limit states' but they may be considered in two groups.

2.5.1 *Ultimate limit states*

Ultimate limit states generally correspond to some form of catastrophic failure (limit states associated with static loading and cyclic loading are dealt with in later chapters of this book).

2.5.2 *Performance limit states*

Performance limit states generally arise when the structure is no longer able to meet all of its performance targets, but is not at risk of catastrophic failure. As an anecdotal example, the houses of people who live in areas where there is heavy snowfall need roofs capable of withstanding large snow loads. Snow loads cause small reversible deformations of the walls, which mean that the door openings change size slightly as the snow builds up. When the doors start sticking, it is therefore time to shovel



Figure 2.6 Westgate Bridge in Melbourne, just after the bridge collapse in 1970 (copyright courtesy of Westgate Bridge memorial committee).

snow off the roof! In this case the snow load may still be well within the limit of catastrophic collapse of the house.

The performance limit state may also simply be based on aesthetic considerations, such as corrosion affecting the appearance of a structure, or its operation becoming excessively noisy.

2.6 Axial and bending stresses

We shall now consider a simple engineered structure such as a bridge, gantry crane or offshore platform that has to withstand static, cyclic and impact loads. The structure comprises a number of different elements such as beams, struts or hollow sections which are joined together, usually by welding. When all the loads are added together, and an individual member is considered, two simple cases can be identified:

Axial loading, where the direction of loading is parallel to the length of the beam (Figure 2.7).

Bending loading, where the direction of loading is at right angles to the length of the beam (Figure 2.8).

In general, the single resultant force acting on the beam from all the sources of loading combined may be neither exactly parallel or at right angles to the length of the beam. In this instance, the force is simply resolved into two equivalent forces;

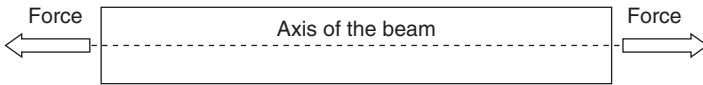


Figure 2.7 Axial loading.

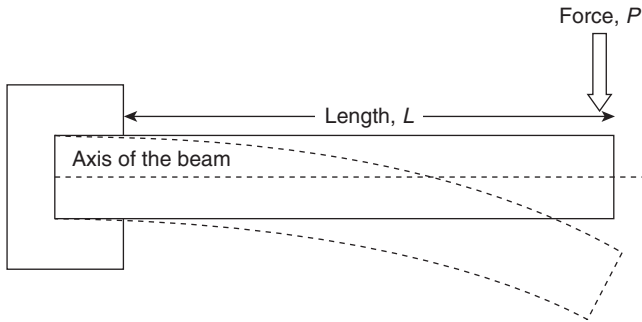


Figure 2.8 Bending loading, showing deflection under loading (dotted line).

one acting along the length of the beam and one at right angles. This particular beam, therefore, experiences both axial and bending loads.

2.6.1 Axial stresses

If the axial load acting on the beam is P and the cross sectional area of the beam is A , then:

$$\text{Axial stress} = \frac{\text{Axial load}}{\text{Cross sectional area}} \quad (2.1)$$

or

$$\sigma = \frac{P}{A} \quad (2.2)$$

where σ is the axial stress and has the units of load divided by area. The SI unit of stress is the Pascal (Pa), which corresponds to one Newton per square metre (N m^{-2}). One Pascal, however, is a very small unit and stress is usually expressed in mega Pascals (MPa), which is a million Pascals or one N mm^{-2} .

Axial stresses may be tensile, when the force is acting in the direction causing the beam to extend (as in Figure 2.7.) or compressive when the force is acting in the

direction causing the beam to contract. By convention, tensile axial stresses are positive, compressive axial stresses are negative.

The action of an axial force such as shown in Figure 2.7 causes the beam to extend a small amount and the strain is defined as:

$$\text{Strain} = \frac{\text{Extension}}{\text{Original length}} \quad (2.3)$$

$$e \text{ or } \varepsilon = \frac{\Delta l}{l} \quad (2.4)$$

Strain, denoted by the symbol e or ε , is a length divided by a length so that its dimensions are metres per metre and, as these cancel out, strain is therefore described as being dimensionless. Strain therefore is simply a number. It can be presented as a decimal number, but as this is usually a very small number much less than one, it is common to show strain as a percentage. For example, a strain of 0.001 is the same as 0.1% strain.

Positive strain corresponds to an extension under positive axial stress (tension), whereas negative strain corresponds to a contraction under negative axial (compressive) stress.

2.6.2 Bending stresses

If we consider a beam, fixed at one end and loaded at the other end at right angles to the length of the beam, as shown in Figure 2.8, then the beam deflects in bending. One example of this form of bending is a diving spring board at a swimming pool. By drawing an analogy with the principle of levers, the load creates a bending moment M in the beam. The value of the bending moment varies along the length of the beam and reaches a maximum at the fixed end. The value of the bending moment M at the fixed end is calculated by multiplying the load P by the beam length L as given by equation 2.5. The units of bending moment are Newton metres, Nm.

$$M = P \times L \quad (2.5)$$

If we take a beam and apply a bending moment, it becomes curved, as shown in Figure 2.9. The original top surface of the beam, AB, becomes the curved top surface, which is longer than the original flat surface, giving a positive strain and a tensile stress. In contrast the lower surface, CD, becomes shorter when curved under bending, giving a negative strain and, therefore, a compressive stress. Somewhere between the upper and lower surfaces there is a plane whose length is unchanged by the bending loading. This plane, which undergoes neither positive nor negative strain, is therefore unstressed and is known as the neutral axis.

For simple, symmetrical beams, the neutral axis is the mid-section of the beam; this is not, however, generally the case and, for non-symmetrical beams, it is necessary to calculate the position of the neutral axis. As we have seen, the neutral axis is the

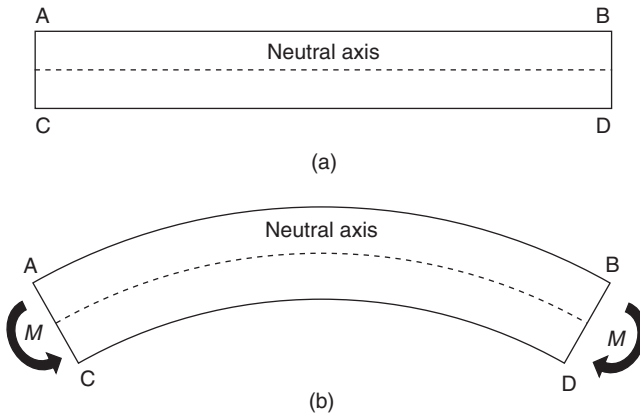


Figure 2.9 A rectangular beam shape shown (a) before and (b) after the application of a bending moment M .

plane in the beam where the stress is zero. The stress at any position in the beam increases as that position moves away from the neutral axis. Using mathematical analysis, it is possible to show that the stress σ at a distance y from the neutral axis of the beam is given by:

$$\sigma = \frac{My}{I} \quad (2.6)$$

where M is the bending moment applied to the beam and I is the second moment of area (also called the moment of inertia) of the beam cross section.

The second moment of area is a parameter used to define the beam's geometrical stiffness and it depends only on the cross-section shape and dimensions of the beam. Standard solutions are available for most beam geometries in engineering textbooks. Equation 2.6, often known as the 'engineer's bending formula', can be used to show schematically how stress varies through the thickness of a beam loaded in bending (Figure 2.10). The engineer's bending formula is also used to calculate the maximum tensile stress at the outer surface of the beam, which generally is the limiting stress condition for the whole beam or weld joint.

2.7 Conclusions

We have identified three major types of loading that structures must withstand: static, cyclic and impact loads. We have seen that applied loads generate stresses in structural members and introduced the concepts of axial stresses and bending stresses, each acting in either tension or compression. In later chapters of this book we consider the

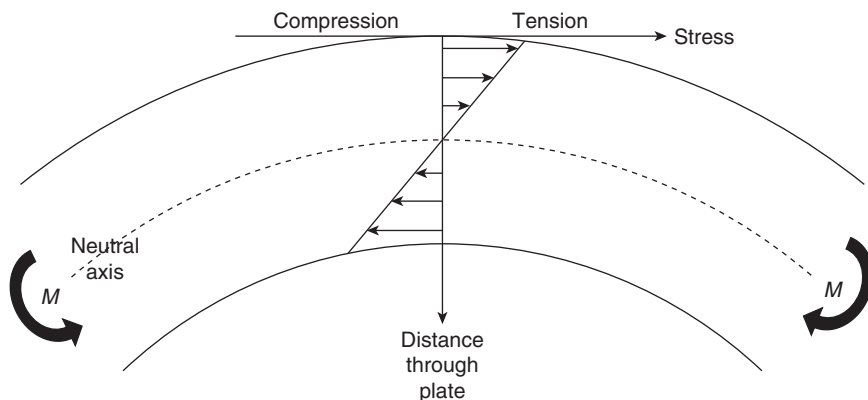


Figure 2.10 Schematic diagram showing the variation of stress through the thickness of a beam loaded in bending.

response of materials to these applied stresses, and describe how limit state design principles are applied for both welded and unwelded materials for static loading and cyclic loading.

2.8 Sources of further information and advice

- Åkesson, B. (2008) *Understanding bridge collapses: from the horizon of the structural engineer*, CRC Press, ISBN-10: 0415436230, ISBN-13: 978-0415436236.
- Cobb, F. (2008) *Structural engineer's pocket book, 2nd edition*, CRC Press, ISBN-10: 0750686863, ISBN-13: 978-0750686860.
- Hicks, J. (1999) *Welded joint design*, Woodhead Publishing, ISBN-10: 1855733862, ISBN-13: 978-1855733862.
- Lawson, D. (2004) *Engineering disasters: lessons to be learned*, Wiley-Blackwell, ISBN-10: 1860584594, ISBN-13: 978-1860584596.
- McKenzie, W. M. C. (2013) *Design of structural elements to Eurocodes*, Palgrave MacMillan, ISBN 978-0-230-21771-3.

3.1 Introduction

Welding is one of the most versatile of joining processes. Solid-state welds have been made for centuries; working two red-hot iron pieces in the forge by striking with a hammer until a joint is made. The ability to join materials is necessary in order to make almost anything, but the joint is often the ‘weak point’ of that structure, where failure is most likely. Creating the strongest, toughest, largest, or most beautiful objects has been made possible by joining metals.

The science of welding began in the early twentieth century with the invention of welding processes such as oxy–acetylene welding and manual metal arc welding (MMA), also called shielded metal arc welding (SMAW) or stick welding. The localised heat source produced from an electric arc or the acetylene flame locally melts steel and fuses two components together along a bond line to make a joint. By the Second World War the development of electric power sources and welding electrodes had made welding a commercial industrial process. However, it was not until the Liberty ship failures during the war that an appreciation of the microstructure and fracture toughness of the steel was sought, and an understanding of how it then behaved when welded was gained. The Liberty ships were welded cargo ships being produced in the USA at an unprecedented rate, but hull cracks and even complete fractures associated with the welds occurred even before some ships had seen service, for (at that time) an unknown reason.

If welding is related to so many failures, one may ask why bother welding at all? The alternative joining methods for steel plates are bolting and riveting. Both involve drilling holes in the plates to be joined, which are overlapped and bolted or riveted together. The structure must be designed to have extra thickness at the joint to compensate for the holes, which coupled with the overlapped joint design means a structure with significant additional thickness and weight. A welded alternative is to simply butt two plates together and perform a full penetration weld. With the development of industrialised welding, structures could be made much thinner, lighter and, therefore, more cheaply. Welding is significantly faster than riveting, as borne out by the production of a welded Liberty ship in a matter of days at its fastest, rather than weeks or months when riveting. The lighter, welded ships could also carry a greater weight of cargo.



Figure 3.1 Welded storage tanks (© TWI).

Today numerous different welding processes have been developed and it is possible to weld a vast number of different engineering alloys from steels to nickel super alloys. Welding quality systems and standards have become established and proven in their effectiveness in ensuring the safety of welding as the joining process for the highest integrity fabrications, such as those found sub-sea, in power stations, pipelines, and storage tanks (Figure 3.1) and in the fastest trains.

Thus, welding represented a huge leap forward in terms of steel fabrication. Nevertheless, welds were still the locations in these structures most susceptible to cracks and other defects, if not properly understood and avoided. Welds are associated with a number of other features such as residual stress and distortion, which are explored in the following sections.

3.2 'Workmanship' defects

Manual welding is a technique which requires significant skill and dexterity. In manual metal arc (MMA) welding (Figure 3.2), the arc is struck between a consumable electrode and the workpiece being welded. The electrode rod is melted at the tip of the arc and metal droplets fall into the weldpool. Thus, as welding progresses the welding electrode becomes shorter. The welder must maintain a constant arc length between the workpiece and the tip of the electrode as the electrode moves along the joint seam, while compensating for the burn-off rate of the electrode. A steady hand to manipulate the electrode is essential, and proper training of welders is necessary in order to achieve satisfactory manual welds. Lack-of-fusion defects may occur in manual welds, particularly in the weld root, where access is most restricted and the



Figure 3.2 Manual metallic arc welding of a steel panel (© TWI).

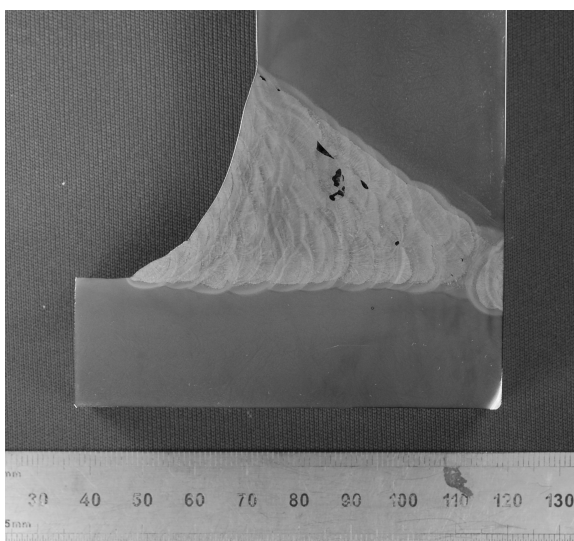


Figure 3.3 Lack of fusion defects and porosity in a multipass GMAW butt weld in carbon manganese steel (© TWI).

weld metal solidifies the fastest, but lack-of-fusion may also occur between weld passes (Figure 3.3). The welder's skill influences the shape, blend and ripples in the weld's surface appearance and the presence of any spatter around the weld. It is possible for pieces of weld slag to be left behind on the surface of the weld metal, even after wire brushing between weld passes, and these can then become trapped as slag inclusions in the joint when later weld passes are laid down over the top.

There are several causes of porosity in welds, and it is a particular problem for aluminium welds. In steels, porosity can be caused by insufficient inert gas shielding of the weld allowing atmospheric gases or moisture to enter the weld pool. In aluminium alloys, porosity is caused by trapped hydrogen which is completely insoluble in the solid state; any grease or moisture in the joint, therefore, causes porosity.

These types of imperfections, which tend to be a result of poor welder skill, are generally known as 'workmanship' defects. It is possible that defects may not affect the structural integrity of the finished weld, but there is usually a limit on the number of these workmanship defects that are permitted, as they can indicate that the welder does not have sufficient skill or experience of that particular welding process. When there is excessive porosity or lack of fusion, the load-bearing cross section of the weld can be significantly reduced. Excessive peaking in the weld cap or excessive penetration of the root pass can result in a high stress concentration at the weld toe. Some lack-of-fusion flaws can be sharp enough to initiate brittle fracture in susceptible steels, so these workmanship flaws cannot be dismissed as insignificant. Limits to the allowable size of defects, known as weld quality levels, are given in standards such as BS EN ISO 5817. Alternatively, their significance can be assessed in a fitness-for-service assessment as described in Chapter 11.

Mechanised welding processes based on gas metal arc welding (GMAW, MIG or MAG welding) reduce the required level of welder skill. The electronic controls in MAG welding self-adjust the length of the welding arc when the torch is moved closer to, or further from the workpiece in the welder's hand. Hence, the size of the resulting weld is much more constant, and the deposition rate of the consumable wire is continuous because it is constantly fed in from a wire feed unit. Mechanised welds can achieve increased welding speeds and have therefore largely replaced MMA welding for the majority of industrial manual welding today (Figure 3.4).



Figure 3.4 Mechanised welding of a gas pipeline girth weld, using a gas metal arc welding and a 'bug-on-band' system to ensure consistent welding (© TWI).

Fully automated welding processes do not require a manual welder to perform them, but instead an operator controls the machine or robot which carries out the welds. The most common example is submerged arc welding (SAW), but laser welding, friction-stir welding and electron-beam welding are also automated welding processes. Automated welding removes welder skill as a factor in the welding quality altogether and performs continuous welds for hours at a time. Even so, mechanised and automated welding processes cannot guarantee defect-free welds every time.

3.3 Weldability

Simply melting and re-solidifying certain alloys does not result in a nicely fused weld. Too much carbon or sulphur in the steel can make weld cracking almost inevitable. The development of better quality steels, made using more controlled steelmaking processes was another necessary step for the success of industrial-scale structural welding. Steel producers today can achieve remarkably high steel strengths, with excellent fracture toughness properties. Complex thermal processing during the rolling and production of the steel are carefully controlled to produce the best quality steel. However, to the metallurgist who has helped develop steel with the most desirable fine grain size and with high yield strength and fracture toughness, welding is the worst thing to do. The melting and solidifying that occurs during welding effectively eliminates the highly designed microstructure and what is produced is weld metal with a coarse, as-cast microstructure. The reduced yield strength of the as-cast weld microstructure can be mitigated by using a welding consumable with an alloy composition that gives the weld metal an equivalent (often higher) strength than the parent metal. This is usually done by adding carbon and/or manganese. The heat input during welding can be controlled to reduce its effect on the parent metal microstructure. However, all welding processes result in a 'heat-affected zone' (HAZ), which is a contour of unmelted parent metal which has nonetheless been affected by the heat of welding such that its microstructure has been altered. This can be seen as a narrow band along the fusion line between the weld and plates in Figure 3.3.

The welding procedure is the 'recipe' for carrying out a particular weld, and is specific to a particular material of a given thickness, welded using a welding process with a certain consumable. The welding procedure specification (WPS) tells the welder at what current and voltage to set the arc welding power source, what size and type of welding consumable to use, and what the joint preparation should be (for instance, the angle of the bevel to allow the welder to see that the root is properly fused). If the combination of all the variables in the WPS are not optimised then, no matter how skilled the welder, the result is a poor quality weld, just as you will not bake a cake by listing any arbitrary combination of ingredients and hoping.

Welding procedures are qualified by making test welds and performing inspection and mechanical tests on the weld to confirm that the weld meets the required quality levels, whilst also having the desired mechanical properties (see 9.2). If the welding parameters have not been chosen correctly it is possible for the weld metal and/or HAZ to suffer from a number of different types of cracking. Any cracking that occurs

as a result of welding is not permitted by welding standards, and results in rejection of the weld and the welding procedure. The ability of an alloy to be welded without becoming susceptible to various kinds of cracking is called its 'weldability'.

3.4 Fabrication cracking in welds

When the weld procedure is wrong, or the welder lacks sufficient skill, a number of welding defects can occur, but the most damaging type of defect to the integrity of the weld are cracks. The term 'crack' to a welding engineer usually means a sharp planar defect with a metallurgical cause, such as hydrogen cracks or solidification cracks. Many welding quality standards differentiate between metallurgical 'cracks' and lack-of-fusion flaws, although the latter may also be planar (and can be just as damaging to the structural integrity). To a welding engineer a 'crack' implies that the weld procedure is inherently flawed, whereas lack-of-fusion defects may simply occur sporadically because of the welder's (lack of) skill. A structural integrity engineer may consider all of these defects as sharp planar flaws and treat them the same way, and can be surprised when a welding engineer objects to being told that a lack-of-fusion is equivalent to a crack. Even within welding engineering care must be taken with terminology.

3.4.1 Hydrogen cracking

For carbon steels, the principal type of weld cracking to avoid is hydrogen cracking (also called cold cracking or sometimes hydrogen-induced cold cracking or HICC), Figure 3.5. Atomic hydrogen is very mobile (i.e. diffuses very quickly) within steel at temperatures above about 250 °C, since hydrogen's small atoms can move easily

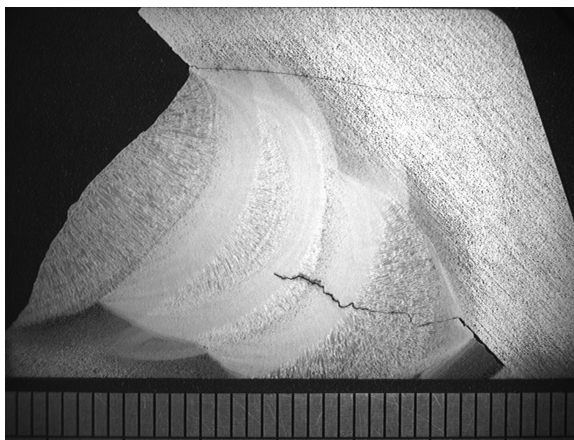


Figure 3.5 Hydrogen crack initiated from the root of a multipass weld (scale bar is in millimetres) (© TWI).

through the gaps in the crystal structure ('lattice') of iron and carbon atoms which make up the steel. Hydrogen can enter steels during welding from the breakdown of hydrocarbons such as oil or grease, or from moisture, either on the parent plate or welding consumables. When the weld then cools to ambient temperature the hydrogen atoms subsequently become trapped in the steel's crystal structure, reducing its ductility and causing it to become embrittled. Under the application of a stress (which can simply be the residual stress from welding) cracking can occur in susceptible microstructures, typically those with high hardness. The critical combination of all four factors (presence of hydrogen, stress, low temperature, and a susceptible microstructure such as one with high hardness) is necessary for hydrogen cracking to occur, and its avoidance is based on removing or reducing one or more of these four factors. The joint inevitably cools to ambient temperature eventually, but by delaying the cooling it is possible to allow any remaining hydrogen to diffuse out of the steel while it is still mobile in the hot steel. Preheating the workpiece slows the rate of cooling, as does applying further heating before the weld has had a chance to cool. Once the steel has cooled down, it can take up to 48 h for hydrogen cracking to occur, so inspection might have to wait some time after welding in order not to 'miss' any hydrogen cracks in the weld.

High hardness is most likely in steels with higher amounts of carbon and manganese. The risk of hydrogen cracking is linked to the carbon equivalent of the steel – a parameter calculated from the steel's composition of all susceptible elements proportionate to their relative contribution to hardness. Steels with medium and high carbon equivalent levels are more 'hardenable', which means they are more likely to form hard phases such as martensite in the HAZ upon rapid cooling. A weld in steel with a carbon equivalent of 0.4–0.5 or above is hard enough to give a significant risk of hydrogen cracking. Hydrogen cracks occur in the location of the weld where the hardness is highest, which can either be the HAZ or the weld metal depending on the type of steel and welding procedure. Welding procedures for medium and high carbon equivalent steels include requirements for preheat, a minimum interpass temperature to maintain between welding passes until the joint is completed, and usually a post-weld heat treatment (PWHT). The PWHT not only helps retard the cooling rate, avoiding martensite formation, and allowing any atomic hydrogen to escape from the steel while at a temperature where it is still mobile, but also reduces the level of residual stress.

For all welding it is important to de-grease plate surfaces, ensure adequate gas shielding of the weld pool and remove moisture. If hydrogen cracking is a likely risk when welding it is possible to select low-hydrogen welding consumables which must be stored in carefully dried conditions (or supplied in vacuum packs). Gas-shielded welded processes are less susceptible to hydrogen cracking than flux-shielded electrodes in MMA welding, which can absorb moisture in the porous flux.

3.4.2 Solidification cracking

The second major type of cracking to avoid when welding carbon steels is solidification cracking (also known as hot cracking), shown in Figure 3.6. This type of cracking

occurs in steels with relatively high levels of impurities such as sulphur and phosphorus, as well as those welds with a susceptible deep and narrow weld cross section profile. Solidification cracking occurs as the weld pool is solidifying, when new metal grains nucleate and grow from the edges of the weld pool toward the centre of the weld. In a perfectly round, hemispherical weldpool, the solidifying grains grow at the same rate from all the outer surfaces of the weldpool and meet at the point in the middle. However, a weldpool is not static, but moves as the welding torch moves, so the hemisphere of molten steel moves along the joint line, and solidifies behind the direction of travel, with the final solidification effectively tracing a line along the middle of the surface of the weld. In sufficiently deep and narrow welds, the solidification fronts do not grow in from the outer edges of a circle, however, but effectively grow as a straight line from the sides of a cylinder. These solidification fronts meet along a plane down the centre of the weld. The last liquid to solidify is iron sulphide, formed from the sulphur in the steel which has a lower melting point than the steel. This iron sulphide then forms along the centreline plane and tries to accommodate all the shrinkage stresses of the weld. However, iron sulphide has poor ductility, thus causing ruptures in the form of a solidification crack along the centreline of the weld.

Deep and narrow weld beads are susceptible to solidification cracking owing to the shape of the solidification front (Figure 3.6). This type of weld profile could be

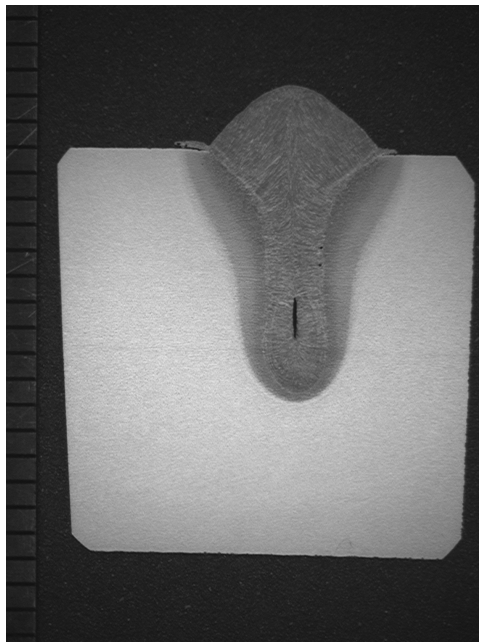


Figure 3.6 Solidification crack along the centreline near the root of a bead-on-plate laser-arc hybrid weld (scale bar is in millimetres) (© TWI).

from high heat input arc welding processes such as GMAW and SAW. Alternatively, 'keyhole' welding processes such as laser welding and electron beam welding usually result in very deep and narrow welds. Susceptible arc weld profiles can be improved by changing the welding procedure to include a larger number of smaller, shallower weld passes.

The avoidance of solidification cracking is also achieved through the reduction of the amounts of sulphur and phosphorus in the parent steel, as well as modification of the weld joint design. Cleaner, more modern, steels have very low sulphur levels and therefore solidification cracking is very rare. However, when steel is sourced from less established suppliers it is possible to find that solidification cracking problems return owing to the presence of sulphur and phosphorus in these steels. A multipass weld with a larger number of shallower weld passes instead of a single deep penetration weld bead is less susceptible to solidification cracking, but imposes a penalty in terms of productivity and weld completion time.

Although solidification cracking is avoidable in principle in steels, it is the most common cracking problem for aluminium alloys. Aluminium alloys do not suffer from hydrogen cracking, nor are impurities in the alloy the culprit when solidification cracking occurs. Instead, it is the deliberately added alloying elements which cause susceptibility. Aluminium alloys solidify from the liquid as crystals of a particular phase, which usually has a different composition from that of the overall alloy. The remaining liquid composition becomes increasingly enriched with the alloying element(s) as solidification progresses. Once the remaining liquid reaches a certain composition called the eutectic composition, all the remaining liquid solidifies at once. If the proportion of liquid remaining at this stage is very small it is dispersed between the main grains and does not cause cracking. Furthermore, if there is a large amount of eutectic liquid solidifying all at once then the volume is enough to accommodate the shrinkage during solidification owing to back-filling the shrinkage cavity. However, if the remaining eutectic is just enough to form a continuous film of liquid along the centreline of the solidifying weld then the shrinkage contraction can be sufficient to rupture the liquid film and cause a solidification crack. Most aluminium alloy consumables have compositions engineered to specifically avoid solidification cracking, provided the correct consumable is chosen for a given aluminium alloy. It is therefore rare to use a matching alloy filler metal to weld an aluminium alloy, as would be the choice for steel.

3.5 Other types of weld defect

In general, weld shape imperfections are often the result of either a lack of welder skill or poor weld fit-up, whereas cracking during welding is often caused by either using an inappropriate welding procedure, choosing the wrong consumable, handling consumables poorly, or contamination in the weld. Once the weld enters service however, it then becomes susceptible to a further set of cracking mechanisms, including fatigue (see Chapters 6 and 7), corrosion, creep, or various kinds of stress corrosion cracking. The susceptibility of a joint to different failure mechanisms

resulting from its in-service conditions can be assessed by mechanical testing. For example, a weld which is to be exposed to potentially corrosive conditions in service can be subjected to a corrosion test in which a piece of weld is exposed to that chemical environment, to see whether cracking occurs or not. Corrosion, creep and fatigue tests that can help the engineer address in-service cracking are described in 9.7, 9.8 and 9.9.

There are other types of fabrication and service cracking associated with welds in specific alloys, such as reheat cracking in the coarse-grained HAZ in Cr–Mo–V steels for high-temperature service; sigma-phase embrittlement in the ferrite phase of stainless steels exposed to elevated temperatures; lamellar tearing in rolled carbon steel plates containing stringer inclusions and through-thickness weld stresses, etc. The welding engineer needs to have a thorough appreciation of all the weldability issues associated with the particular alloy being welded and of its intended service conditions. However, the intention of this book is not to be a compendium of weld cracks, but to give an indication of the most common kinds of weld defects and their significance in terms of fracture and fatigue.

All cracks and planar defects are significant when considering the resistance of a welded structure to fracture and fatigue. These defects are the most likely initiation site for a brittle fracture, or for the propagation of a fatigue crack and, therefore, an understanding of the causes of possible weld flaws is the first stage in avoidance of failure.

3.6 Welding residual stresses

Even in nominally 'defect-free' welds, there is still a hidden source of concern for the structural integrity engineer. The heating and cooling of the weld pool and surrounding material is associated with thermal expansion and then contraction, but the small weld pool is constrained within the solid parent material and thus the localised thermal expansion and contraction cannot happen freely. This is illustrated schematically in Figure 3.7a, where the weld metal which has just solidified, is hot and takes up the same volume as the weld groove. If this weld metal could cool down in an unrestrained way it would contract owing to thermal shrinkage (Figure 3.7b). However, the weld metal is not able to cool in an unrestrained way, because the surrounding parent metal prevents this contraction. As the weld metal cools down, it is not allowed to shrink and therefore high tensile residual stresses are introduced in the weld metal (Figure 3.7c). These residual stresses remain after the joint is completed and can be as high as the yield strength of the parent material. Any stress greater than this causes localised yielding of the material which then limits the residual stress to the yield strength. The actual residual stress distribution varies through the thickness of the weld, and is different along the length of the weld and across the weld. The tensile residual stresses near the surface of the weld are balanced by compressive residual stresses elsewhere in the joint. The residual stress distribution also depends on the weld size and the heat input used for welding. Therefore making any assumptions about the residual stresses locked inside a welded joint can be very tricky and it is

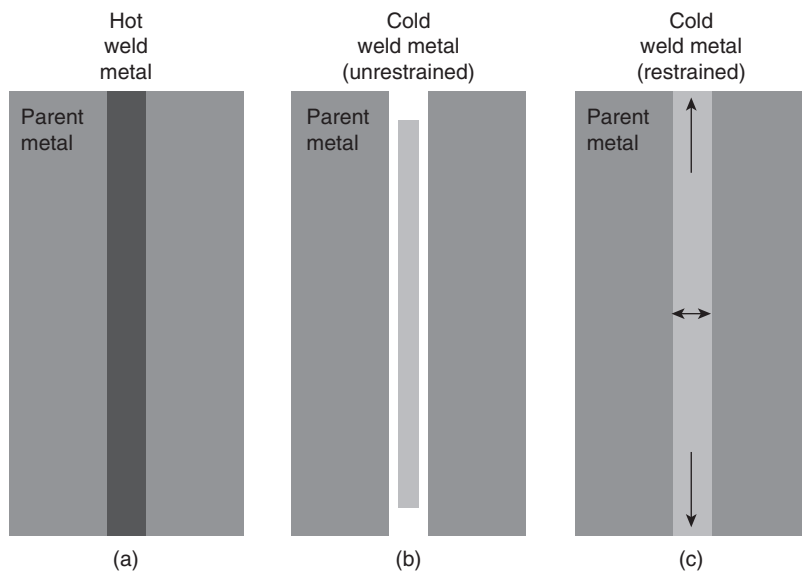


Figure 3.7 Schematic diagram of residual stresses resulting from differential thermal expansion and contraction between the weld metal and surrounding parent metal.

often simply assumed that the residual stresses in the vicinity of an as-welded joint are tensile and of yield magnitude.

Although there are a number of techniques that can measure residual stresses, many of them are destructive techniques that can be used on test welds, but not on welds in-service. Some of the simpler methods to apply, such as centre hole drilling, only measure the residual stress at the plate surface, giving no information about the residual stresses in the centre of the plate. The most accurate methods, such as neutron diffraction, are very expensive research techniques which are not applied commercially. Usually, it is necessary to make some conservative assumptions about the residual stresses in a weld joint when making calculations about the effect the residual stresses might have on the joint's resistance to fracture or fatigue (see Chapters 11 and 12).

Residual stresses alone can be enough to cause brittle fracture in steels with sufficiently low fracture toughness. This was the then unknown cause of many of the Liberty ship failures that cracked without the ships seeing any seaborne service. Where brittle fracture is a risk, particularly for steel greater than 25 mm thick, then post-weld heat treatment (PWHT, also called stress relief) is routinely required in order to bring the residual stresses down to 20–40% of the yield strength from as high as yield magnitude. For carbon manganese steels, PWHT is usually performed at a temperature of 550–625 °C and held for around an 'hour per inch' (25 mm) of plate thickness, in addition to a controlled heating and cooling period. The residual stresses associated with welds can also make so-called 'repair' welds critical

to the integrity of an overall structure, because it is often difficult to control PWHT of repair welds, if PWHT is performed at all. Repair welds can leave yield strength level residual stresses (and defects) in a location which had previously been stress relieved. This could be more damaging to the structural integrity of the structure than simply leaving the existing flaw alone. These problems are discussed further in Chapter 11.

3.7 Distortion

The thermal expansion and contraction stresses in the weld (which are initially greater than the yield strength of the steel) cause localised yielding to occur around the weld. In some situations, permanent and significant deformation may result from this localised yielding at the weld, known as weld distortion. In thinner section material and sheet metal, or in alloys with a low yield strength or large thermal expansion coefficient (such as aluminium and stainless steel), the residual stresses are sufficient to deform the whole sheet and significantly distort the welded structure.

Weld distortion can present itself in a number of ways. Simple shrinkage of the weld metal, both along the weld, and transverse to the weld alters the dimensions of the final welded component. Where this shrinkage is not uniform through the thickness of the plate, angular distortion may be introduced where the two sides of a butt weld bend upwards, or a fillet welded attachment moves away from vertical. Welding all round a plate can cause the plate to become concave. The welded stiffeners on the inside of the panels which form ship hulls are a typical example in which the stiffener welds can cause ripples or bowing of panels along the length of the ship; these ripples can be seen in the hull of the *Glovis Champion* vehicle carrier seen in Figure 3.8.

The buckling of welded structures can significantly affect their performance as well as their appearance. Buckling occurs when the distortion of a structure under loading causes the beam or member to distort in an unstable elastic and unrecoverable manner. A welded structure which has suffered from weld distortion no longer has the most rigid dimensional shape, and a distorted beam is susceptible to an unstable buckling condition under much lower loading than a beam without the weld distortion. Most steels have high stiffness and do not deform significantly under load, so weld distortion tends to be a cosmetic concern rather than a safety issue. However, for aluminium structures which have inherently lower stiffness than steels, but are also susceptible to weld distortion, buckling resistance is a main design failure mode, and excessive distortion associated with welds can cause a component to be scrapped. Even in steel structures, remedying distortion can form a major cost of a welding project.

Distortion can be avoided or reduced by using jigs and fixtures for welding, but this may introduce higher residual stresses. Plates can be pre-set at an angle equal to the expected distortion so that the welding subsequently brings them flat. Correcting distortion is mainly based on localised pressing or hammering to restore the correct dimensions to the component. It is also possible to use thermal techniques such as



Figure 3.8 The *Glovis Champion* vehicle carrier (built in 2013) showing ripples and distortion of the lower hull panels (see magnified portion of hull underneath the ‘OV’ of ‘GLOVIS’) caused by welding (© TWI).

local flame heating to introduce deformation to oppose the weld distortions. There is great skill in applying thermal techniques correctly so that the required shape is achieved without damaging the mechanical properties of the structure.

3.8 Conclusions

Welding is a process which relies on combining a weldable material, the correct set of welding parameters, and sufficient welder skill to achieve a good weld. The number of factors involved means there is plenty of scope for errors to occur. When structures fail, the location of failure initiation is often the weld, because defects in welds, and the associated residual stresses at welds, can cause either fracture or fatigue. This is why it is important for welding engineers to understand these failure mechanisms, and be able to take action during design and fabrication to avoid catastrophic failure before it is too late.

3.9 Sources of further information and advice

BS EN ISO 5817 (2007) *Welding. Fusion-welded joints in steel, nickel, titanium and their alloys (beam welding excluded). Quality levels for imperfections*, British Standards Institution.

Cheng, Y. F. (2013) *Stress corrosion cracking of pipelines*, Wiley–Blackwell, ISBN-10: 111802267X, ISBN-13: 978-1118022672.

- Delatte, N. J. (2009) *Beyond failure: forensic case studies for civil engineers*, ASCE Publications, ISBN-10: 0784409730, ISBN-13: 9780784409732.
- Houldcroft, P. T. (1977) *Welding Process Technology*, Cambridge University Press, ISBN-10: 0521215307, ISBN-13: 978-0521215305.
- Singh, R. (2008) *Weld Cracking in Ferrous Alloys*, Woodhead Publishing Ltd, ISBN-10: 1845693000, ISBN-13: 978-1845693008.

4.1 Introduction

In Chapter 2 we considered typical sources of load acting on a structure and grouped them into three categories; static, dynamic and impact. In this chapter we consider the response of a material when it is loaded statically, and show how a limiting state is defined and used for design purposes. First, we consider unwelded material, then show how fairly simple principles can be used to design welded joints for static loading.

4.2 Load–extension curves

The response of material when a very slowly increasing load (to all intents and purposes a static load) is applied was first studied in the 17th century. Simple experiments were carried out which involved applying a load and measuring the length by which the material increased. The results can be plotted as load against extension (Figure 4.1). Initially, as the load is increased the graph is a straight line and the extension is proportional to the load. In this region, when the load is reduced the extension also reduces so that the material length is always the same as it was under the same load when the load was increasing. Furthermore, if the applied load is completely removed then the material returns to its original length, i.e. at zero load there is zero extension.

The behaviour of the material in the region O to A is known as elastic behaviour, i.e. the material ‘springs back’ completely when the load is removed. Furthermore, in this linear region, extension is proportional to load; this is known as Hooke’s Law. As the load is increased further, beyond A, a major change occurs. The load–extension curve deviates from a straight line, and the extension for a given load is greater than that anticipated by Hooke’s Law. If the load is increased to point B before being reduced, the unloading response of the material corresponds to the path B–C. The line B–C is parallel to the elastic region but when the load is completely removed at C there is a small remaining extension of size O–C. This is a permanent plastic deformation.

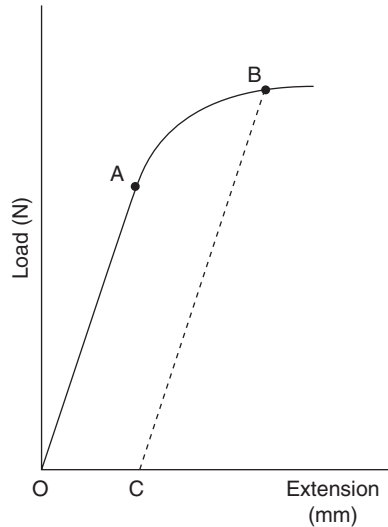


Figure 4.1 Idealised load versus extension graph for steel under tension.

Thus the response of the material can be divided into two regions; elastic behaviour up to point A where Hooke's Law is obeyed and unloading restores the original length, and the plastic behaviour regime beyond point A where Hooke's Law is no longer valid and, on total removal of loading a small permanent deformation remains.

4.3 Stress–strain curves

The load–extension diagram (Figure 4.1) is very useful in appreciating behaviour, but is not helpful from an engineering design point of view. If the sample were twice the length then we would expect twice the extension for the same load. Furthermore, if the sample were thicker, then we would expect smaller extensions for the same load. By using stress in place of load, and strain in place of extension (as defined in section 2.6.1), these geometry effects are eliminated and the stress–strain curve corresponds to a consistent representation of material behaviour.

4.3.1 *Young's modulus*

Because stress–strain curves are obtained from load–extension curves, their appearances are very similar; a typical stress–strain curve for steel is shown in Figure 4.2. There is a linear region up to the limit of elasticity where stress is proportional to strain. This statement is simply a re-phrasing of Hooke's law that extension is proportional to load. The slope of the linear portion is called the modulus of elasticity, also known as Young's modulus, E , where $E = \text{stress/strain}$.

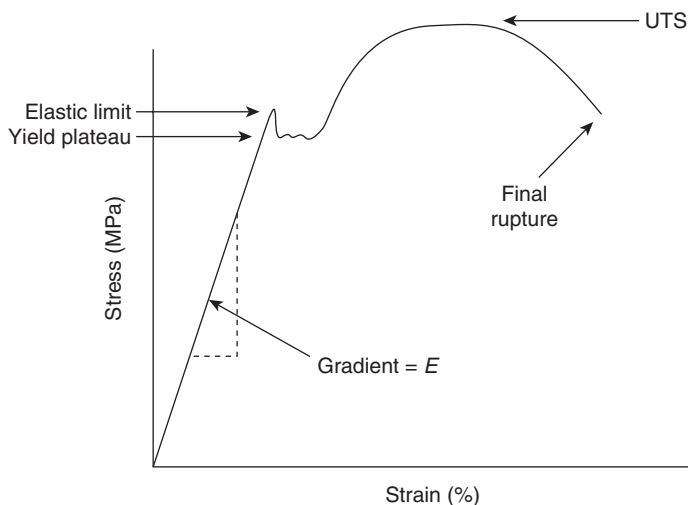


Figure 4.2 Typical shape of a stress–strain graph for a structural steel, showing the definitions of the yield point, the ultimate tensile strength (UTS), and Young’s modulus (E).

Young’s modulus is a measure of the stiffness of a material. It has the same units as stress, and can be expressed in N mm^{-2} or MPa, but is also commonly given units of GPa (equal to a thousand MPa). A typical value of Young’s modulus for steel is 207 GPa; Young’s modulus for aluminium is approximately 70 GPa. Steel is therefore about three times stiffer than aluminium; in other words, under the same applied stress aluminium extends elastically about three times as much as steel.

4.3.2 Yield strength

The stress–strain graph for steel, Figure 4.2, shows that as the stress is increased the graph deviates from the linear region and in many steels a sharp but relatively small drop is seen. This point is the elastic limit and corresponds to the onset of yielding in the steel i.e. when the atoms in the steel start to slide over each other. This sharply defined point is known as the steel yield strength, and is given the symbol R_e in EN specifications.

After yielding and a sharp fall in stress, there is a flat region in Figure 4.2 called the yield plateau or Lüders plateau; this is also known as the lower yield point. This is caused by the atoms sliding over each other owing to the movement of dislocations. In many steels, these dislocations are ‘pinned’ and a high stress is required to make them move (which gives an initial ‘upper yield point’). Once they are moving, however, it requires a lower stress to continue their movement. As they continue to move, they become entangled with each other and increasingly high stresses are required for their continued movement beyond the yield plateau.

Some materials, including austenitic stainless steels and aluminium, do not show a clearly defined yield point. The main reason for this difference in behaviour is

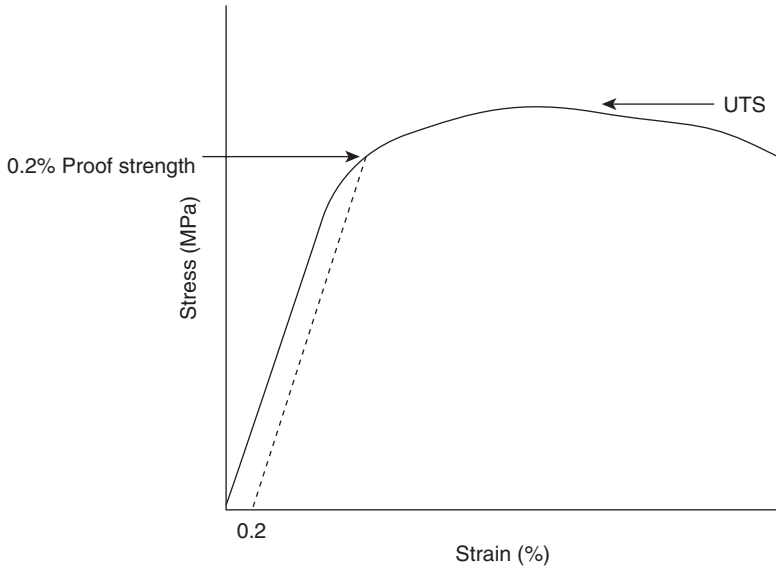


Figure 4.3 Definitions of yield parameters for a stress–strain curve that does not show a defined yield point.

that in these materials dislocations are not initially pinned and there is nothing to prevent the material deforming freely. A typical stress–strain graph for these materials is shown in Figure 4.3. There is a gradual deviation from linear behaviour and it is therefore difficult to determine the exact point at which deviation occurs. To overcome this, an arbitrary but useful agreed definition of yield is used, normally the 0.2% proof strength. This is the stress corresponding to a permanent strain of 0.2%, or 0.002. It is determined by taking a strain of 0.2% at zero load and constructing a line upwards, parallel to the elastic part of the graph, Figure 4.3. The stress where this line crosses the measured stress–strain graph is the 0.2% proof strength, also known as $R_{p0.2}$.

Another parameter that can be obtained from any stress–strain curve is the stress corresponding to a total strain of 0.5% (i.e. 0.005). This is used in some standards and is known as the 0.5% offset strength.

4.3.3 Ultimate tensile strength

After yielding, the stress–strain graph for most materials continues upwards as plastic deformation continues, as seen in Figures 4.2 and 4.3. Eventually, a maximum is reached, where the stress is known as the ultimate tensile strength (UTS), and after this point the stress decreases with increasing strain.

At first it is difficult to understand why stress should decrease. Observing a tensile test shows that immediately after the (UTS) is reached, plastic deformation no longer

occurs evenly in the test piece but becomes concentrated in one location. At this location, the cross section area reduces very rapidly, giving rise to ‘necking’ (see 9.4). However, the stress used to plot the stress–strain graph is based on the original cross section area, not the much smaller area that is actually deforming. It is this simple definition of the stress (also known as the ‘engineering stress’) that gives rise to the apparently strange appearance of the stress–strain curve.

If the true stress (i.e. the load divided by the instantaneous actual cross section area) and true strain (extension divided by the instantaneous length) were used to describe the behaviour of the necked region, then a continuously increasing stress–strain straight-line graph would be obtained which flattens after reaching the UTS. Measuring true stress and strain properties is more difficult than measuring engineering stress–strain properties. However, sometimes true stress–strain properties are required for some purposes such as finite element modelling of material behaviour. In this book, engineering definitions of stress and strain are used.

4.4 Static limit state design

The formal assessment of static loading using limit state design principles is fairly complex and involves mathematics that are beyond the scope of this book. Under static tensile loading we generally wish to ensure that plastic deformation does not occur. In other words, when the load is removed, the structure regains its original shape and dimensions. This is equivalent to ensuring that the applied load does not cause the stress in the material to exceed its yield strength.

When carrying out the design process there may, however, be a number of uncertainties about quantifying all the sources of loading, and a safety factor is therefore introduced to account for these. The maximum design stress is therefore given by:

$$\sigma_{\text{design}} = \frac{\sigma_y}{\text{safety factor}}$$

The precise value of the safety factor to be used depends on the specific application and structural code, but we assume a safety factor of 1.5. This is the same as saying that the maximum stress in the structure must not exceed 2/3 of the material yield strength. Sometimes additional safety factors are applied, particularly if the welds are not subjected to 100% non-destructive inspection. These are known as joint factors, which will be equal to 1.0 for a fully inspected joint and increase as the extent of the weld inspection reduces.

Thus far, we have been considering only the parent material itself and not the properties of the welded zone. We have emphasised that mechanical properties such as yield strength are strongly dependent on the material chemical composition and heat treatment. The yield strength of the weld metal may, therefore, be very different from the parent steel and depends on the welding consumable and welding procedure

employed. Consumable manufacturers have developed very sophisticated techniques to achieve guaranteed weld metal properties and, for most practical purposes, it is possible to use a welding procedure that ensures the yield strength of the weld metal exceeds that of the parent steel, that is the weld metal overmatches the parent steel. The term 'overmatched' should be used with care, however, as sometimes it can be taken to mean that the consumable has a greater alloy content than the parent steel.

The heat affected zone (HAZ) of a weldment has been subjected to a severe thermal cycle and, consequently, there is a range of microstructures in the HAZ. Each of these individual microstructures exhibits a different yield strength. Direct measurement of the yield strength of a particular HAZ microstructure is difficult, but by measuring the hardness of local regions using an indentation technique (such as described in 9.11), estimates of the local material strength can be made. For most structural steels and welding procedures, the yield strength of all regions of the HAZ is greater than that of the parent metal.

In many instances, therefore, all regions of the weld joint overmatch the strength of the parent material. When the weld joint exhibits strength overmatching, then the structure's design stresses can conservatively be based on the parent material yield strength. There are some circumstances, however, when the yield strength of all parts of the weld zone may not be greater than that of the parent material. For steels which achieve a high parent material yield strength through high alloying content and/or complex heat treatment, such as high-strength pipeline-grade steels, it may not be possible to develop consumables with matching yield strength. Furthermore, the HAZ may be softer (i.e. lower hardness and hence lower yield strength) than the parent material. Specialist design approaches are used in these cases. The second main example of strength under-matching welds is in aluminium alloys which are work hardened by rolling or heat treated to achieve high parent metal yield strength. In these alloys, the welding thermal cycle destroys the carefully prepared parent metal fine-grain structure so that both the weld metal and the HAZ show microstructural grain growth and, therefore, may be softer than the parent material. A second heat treatment may be used to restore some of the joint strength in heat-treatable aluminium alloys, but the strength loss in the HAZ of work-hardenable aluminium alloys is generally unrecoverable.

Where the weld or HAZ strength under-matches that of the parent metal, then the design stresses must be based on the strength of the weakest part of the joint, i.e. the region with the lowest yield strength.

4.5 Conclusions

In simple terms, satisfactory static design is achieved by ensuring that the material yield strength is not exceeded. The application of this principle to welded joints is relatively straightforward in that, for most structural steels, the weld can be made such that all regions of the weld zone have greater yield strength than the parent material. We have taken a very simple approach: in some structures a certain degree of plastic deformation is permitted under bending loading. Furthermore, we have not

considered either excessive deformation or elastic buckling both of which could provide limiting states.

4.6 Sources of further information and advice

- Bruneau, M., Uang, C-M. and Sabelli, R. (2011) *Ductile design of steel structures, 2nd edition*, McGraw-Hill Professional, ISBN-10: 0071623957, ISBN-13: 978-0071623957.
- Mathers, G. (2011) *The welding of aluminium and its alloys*, Woodhead Publishing, Ltd. ISBN-10: 1855735679, ISBN-13: 978-1855735675.
- McKenzie, W. M. C. (2013) *Design of structural elements to Eurocodes, 2nd edition*, Palgrave Macmillan; ISBN-10: 0230217710, ISBN-13: 978-0230217713.

Brittle fracture and the behaviour of cracks in structures

5.1 Introduction

We now consider in more detail the behaviour and failure of a material when it is subjected to a static load. The response of the material under load is represented by the stress–strain curve (see 4.3); failure (usually known as fracture) eventually occurs at the fracture stress. The resistance of a material to fracture is known as its toughness. There are many different parameters used to measure toughness. One very simple parameter is the energy required to cause fracture in a tensile test. This is represented by the total area under the stress–strain curve up to fracture. Later in this chapter, we explore other energy-based measures of toughness such as the Charpy impact energy and the J integral.

If a material fails in a tensile test after absorbing a great deal of energy (i.e. the material is relatively tough) then the failure mechanism is known as ductile failure or ductile fracture. Brittle fracture is the term given to failure when only a small amount of energy is absorbed, i.e. the material possesses relatively poor toughness. The resistance of a material to unstable fracture is known as its fracture toughness, and is measured by fracture toughness tests.

5.2 Nature of brittle fracture

Studies of brittle fracture have shown that failure occurs by the initiation of a crack, usually at a notch or stress raiser. The crack then propagates extremely rapidly across the load-bearing area at a velocity approaching the speed of sound in the material. In some instances, the running brittle crack may stop or arrest, particularly if it grows into a region of lower stress or higher toughness material. The brittle fracture usually initiates at a single point (at the notch or stress raiser) and, therefore, the material toughness and microstructure at this particular location is of vital importance.

In structures, brittle fracture can occur without warning as there is usually no prior plastic deformation at the initiation of fracture. The first evidence of a brittle fracture may be when a plate cracks or a pressure vessel explodes. Furthermore, because the brittle crack propagates very rapidly across the load-bearing member, catastrophic failure often results in loss of life and significant damage to equipment. Welded joints

are particularly susceptible to brittle fracture because they are inevitably associated with a stress concentration and the complex microstructures in the vicinity of the joint may contain a region of very poor toughness even if the parent material has good toughness.

5.3 The three factors for brittle fracture

Unlike ductile failure which is caused simply by excessive loading on a structure to beyond its ultimate tensile strength (UTS), brittle fracture requires a critical combination of three factors. These are; a sufficiently low toughness material, a stress, and a flaw (or other stress concentration). These three factors can be thought of as three corners of a triangle, as illustrated in Figure 5.1. The way these three factors interact to cause brittle fracture is described in the fracture mechanics models discussed later in this chapter, but it is useful to look at each factor in turn to consider where they arise in engineering structures.

5.3.1 Weld flaws

Welds, in addition to presenting a stress concentration at weld toes, may also contain a flaw or crack. The types of flaws that most commonly initiate brittle fracture are the typical welding defects, such as lack-of-fusion flaws or hydrogen cracking (discussed in Chapter 3) or a fatigue crack that has grown sufficiently large (see Chapter 6). A flaw or crack increases stress and is a very likely point at which brittle fracture may initiate. In unwelded structures the stress concentration might be a bolt hole or other stress raiser.

5.3.2 Weld stress

Brittle fracture is perhaps of most concern in welded structures because a residual stress alone (see Chapter 3) is sufficient to propagate a brittle fracture, without any external loading. In a thick plate containing a butt weld, if it is left alone, the butt weld will not exhibit a ductile failure because it cannot deform and yield without an

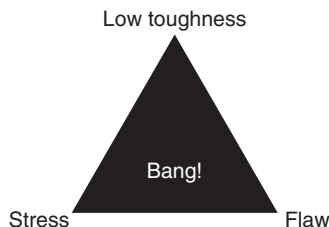


Figure 5.1 Triangle illustrating the three factors required for brittle fracture.

external applied load. However, a welding defect in such a plate may initiate brittle fracture with no external load, because of the effect of the welding residual stress alone. An example of this occurred in some of the Liberty ship failures where ships fractured after fabrication without leaving dry dock. This makes the 'stress' corner of the triangle in Figure 5.1 of particular concern, and the requirement for post-weld heat treatment (PWHT) of welded structures to reduce the residual stresses is therefore very important for brittle fracture avoidance. Any applied external loading and stress concentrations in the structure's design also contribute to the risk of brittle fracture.

5.3.3 Low toughness

The low toughness factor becomes important in a number of different instances. The main set of materials at risk of brittle fracture are ferritic steels, because they have a crystal structure which shows a ductile-to-brittle transition at a certain temperature (described in 5.4). Austenitic type metals, such as austenitic stainless steel or aluminium alloys, do not show a transition between ductile and brittle behaviour with temperature in this way. However, the ductile-to-brittle transition is not the only cause of low fracture toughness. We have already seen that yield strength is strongly dependent on microstructure; toughness is also very dependent on microstructure. The influence of microstructure can be seen in low heat input welds in steels with a moderately high carbon equivalent by the formation in the HAZ of martensite which has low toughness. In high heat input welds, the coarse grains formed in the HAZ also result in low fracture toughness. Reduction in fracture toughness also occurs as a result of embrittling mechanisms such as strain ageing in carbon steels or sigma-phase formation in stainless steels. Adding even a small amount of nickel to a steel improves its fracture toughness. The only steel microstructure which is both strong and tough is a fine grain structure; this is how high-strength low-alloy steels achieve this optimum combination of mechanical properties. When welding fine-grained steels, the heat input and welding procedure play a crucial part in the potential reduction in fracture toughness within the HAZ and weld metal and, therefore, need to be closely controlled.

5.4 Ductile-to-brittle transition

The striking effect of temperature on the fracture toughness of ferritic steels is shown in the ductile-to-brittle transition plotted in Figure 5.2. In a typical toughness transition curve, the shape is independent of the parameter used to characterise the toughness. The curve is S-shaped, with an upper shelf which defines the ductile behaviour of the steel, and a lower shelf where the material is brittle. In between there is a transition region where the toughness can be anywhere in between, and the toughness when measured in this region shows a significant amount of inherent scatter in the results obtained. The ductile-to-brittle transition shown in Figure 5.2 does not show scale markers on the axes for a deliberate reason: different steels show the ductile-to-brittle transition at different temperatures, and, in addition, the steepness of the transition

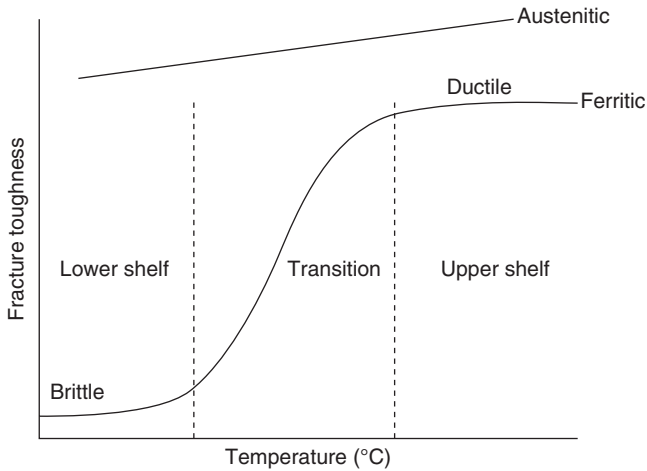


Figure 5.2 Schematic ductile to brittle transition curve for ferritic steels, defining the upper shelf, lower shelf and transition region. Note that austenitic materials do not show this type of toughness transition with temperature.

and the fracture toughness on the upper shelf vary with the steel. An old steel with a high sulphur level may have a transition temperature only just below room temperature and a very low upper shelf toughness, whereas a modern fine-grained steel might be on the upper shelf well below -50°C and have a high fracture toughness on the upper shelf. The ductile to brittle transition only occurs for ferritic materials, and an austenitic type material (such as aluminium alloys, austenitic stainless steels or nickel alloys) do not show a marked reduction in fracture toughness at low temperature.

Toughness, however, is not an absolute material property and depends on the conditions of the test as well as the temperature. In any material the following factors give rise to lower measures of toughness:

- an increase in the rate of applying the load, i.e. increasing the strain rate;
- the sharpness of a stress raiser, crack or notch;
- an increase in the material thickness.

The ductile-to-brittle transition curve can be generated from fracture toughness tests or Charpy tests over a range of temperatures. These mechanical tests are described in more detail in Chapter 9. It is more common to generate the transition curve using Charpy testing, typically from ten data points (Figure 5.3). Although the transition temperatures determined from Charpy or fracture toughness tests do not exactly correspond, the large amount of experience in Charpy testing means that impact requirements are commonly used for qualifying welds for low-temperature service. If a material exceeds a given Charpy impact energy value (usually 27 or 40 J) at a temperature a given amount lower than the lowest service temperature, then experience shows that it can be considered to be at a low risk of brittle fracture. An impact energy value of 27 or 40 J corresponds to the part of the Charpy transition curve where the

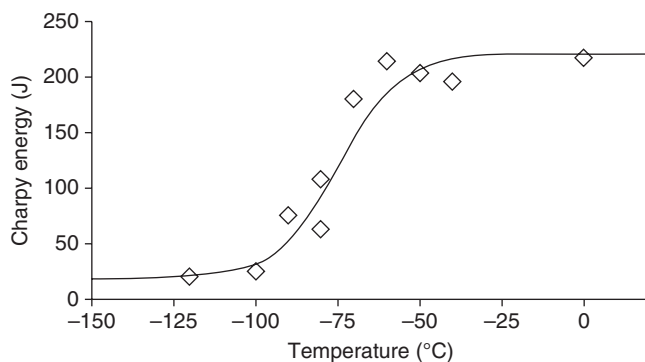


Figure 5.3 Charpy transition curve fitted to ten Charpy test data points tested over a range of temperatures. In this steel the midpoint of the ductile to brittle transition is at -75°C and the 27 J Charpy energy corresponds to a temperature of -105°C .

lower shelf is starting to become the transition region. Understanding the behaviour of a steel (or weld) in terms of its ductile-to-brittle transition curve can ensure that it is designed so that it is never operating at temperatures corresponding to the lower shelf. It is important to carry out toughness testing at the lowest anticipated temperature, otherwise a slight drop in temperature below this may cause a potentially catastrophic reduction in fracture toughness.

5.5 Welding and fabrication codes

Brittle fracture is a particular risk for welded structures because welds are the regions where all three factors for brittle fracture are most likely to occur. The residual stresses associated with welding can be as high as yield magnitude tensile stress in the as-welded condition. Defects associated with welds include lack of fusion, porosity, cracks, slag inclusions, lack of penetration, and any of these could act as the initiation for a brittle fracture. Low fracture toughness microstructures can also occur within the HAZ of a weld owing to the weld heat input being too low or too high.

To prevent brittle fracture occurring in service, most fabrication codes have a common approach and similar sets of rules when designing and fabricating steel structures. Examples of these fabrication codes are BS PD 5500 (pressure vessels), BS EN 1990 (steel structures), BS EN 1993 (steel buildings), American Bureau of Shipping (ABS) rules (fixed offshore structures), BS 4515 (pipelines). The rules within these established codes and standards are usually sufficient to avoid brittle fracture.

The first requirement is for Charpy testing if the structure operates at below ambient temperature. A minimum Charpy energy of 27 or 40 J (for example) must be achieved at a test temperature a given amount lower than the minimum service temperature. The temperature reduction, below the minimum service temperature at

which the test must be carried out, is usually specified by the fabrication code. The various microstructures in the weld zone are investigated by testing sets of Charpy specimens notched into the parent metal, the weld metal and various positions within the HAZ such as the fusion line (FL), and FL + 2 mm or FL + 5 mm, moving further away from the weld. Meeting the code's Charpy requirements gives confidence that the material has sufficient fracture toughness at the given minimum service temperature.

5.5.1 Post-weld heat treatment (PWHT)

The codes give advice on suitable designs in order to prevent high stress and do not permit certain features that could act as stress raisers. For thick-section steel (thicker than 25–40 mm depending on the code), welds have to have a post-weld heat treatment (PWHT) owing to the constraint in thick-section material that makes brittle fracture more likely (see 5.6.5). The PWHT, if performed correctly, may reduce the level of residual stress from as high as the yield strength of the steel down to typically 20–40% of the yield strength. The PWHT includes specific heating and cooling rates, and the welds are held at a temperature of around 550–625 °C for typically an hour per inch of thickness.

After fabrication of pressure vessels or pipes, a proof load may be applied so that the structure is pressurised to beyond the loading to be experienced in service. The intention is to cause localised plastic deformation at stress raisers, which, upon unloading, experience a reduced residual stress. The test also provides confidence that the structure containing any existing fabrication flaws is safe from brittle fracture under the given proof test conditions, which are designed to be more severe than those expected in service.

5.5.2 Avoiding flaws

The level of non-destructive testing (NDT) and inspection required is often specified by fabrication codes, depending on the importance of the welds. NDT and inspection is covered in more detail in chapter 10. Limits to the allowable flaw sizes for different types of weld imperfection are given in the fabrication standards. Setting maximum allowable flaw sizes for the inspection restricts the size of any potential crack that could initiate a brittle fracture. Most fabrication codes specify that cracks are not allowed, to avoid the risk of brittle fracture; in practice the loss of load-bearing cross section owing to weld defects is usually fairly negligible. Code requirements are generally good engineering common sense, but it is useful to understand why certain requirements are included and what their role is in keeping structures safe, in terms of the triangle of brittle fracture risk.

It may not always be possible to meet the full requirements of a fabrication code or standard: the Charpy impact toughness may be slightly lower than the minimum requirement or the strength may be less than the specified minimum. Defects greater than those allowed in the fabrication code may have been detected in a completed structure, where repair might not be realistic and scrapping too costly. Making sure

that brittle fracture is still avoided in these situations requires a fitness-for-service assessment, based on fracture mechanics calculations and fracture toughness data. Such assessment is discussed in detail in chapter 11.

5.6 Principles of fracture mechanics

Fracture mechanics is a relatively recent science with its first developments based on experiments on brittle materials (such as glass) during the First World War by A. A. Griffiths, working as an aircraft engineer in the UK. At that time it was a mystery why materials failed at a stress that was orders of magnitude lower than that predicted by the sum of the strengths of the individual atomic bonds. Griffiths showed that materials contained microscopic cracks, and that failure occurred by the extension of these cracks. The groundwork of Griffiths in brittle materials was further developed in the 1950's by G. R. Irwin and colleagues at the US Naval Research Labs in Washington, D.C., when more ductile materials, such as steels, were being considered after the Second World War.

5.6.1 Cracks under load

The behaviour of a crack in a structure is dictated by the tiny region at the very tip of the crack, and the stresses that are experienced there. A crack acts as a very high stress concentration, with the stresses at the crack tip being many times greater than the bulk stress applied to the whole structure. To consider the stress concentration effect of a crack it is useful to start with a round notch and define the stress concentration factor as the maximum stress experienced as a consequence of the notch. For example, the stress concentration effect of a round hole is three times the applied stress. As the round hole becomes more squashed and elongated to become a longer thinner flaw, the radius of the notch tip becomes smaller, and as it becomes sharper the stress concentration effect becomes larger (Figure 5.4). This can be explained in mathematical terms by considering an elliptical notch through a very large sheet of material (Figure 5.4). The flaw, which has length, $2a$ and depth $2b$ and the radius at the tip is r , is under an applied axial stress σ .

The stress at the tip of the flaw is given by equation 5.1.

$$\sigma_{\text{tip}} = \sigma \left(1 + 2\sqrt{\left(\frac{a}{r}\right)} \right) \quad (5.1)$$

where the radius is:

$$r = b^2/a \quad (5.2)$$

For a sharp crack where the length of the defect ($2a$) is much greater than the height of the crack ($2b$), equation 5.1 can be simplified to:

$$\sigma_{\text{tip}} = 2\sigma\sqrt{\left(\frac{a}{r}\right)} \quad (5.3)$$

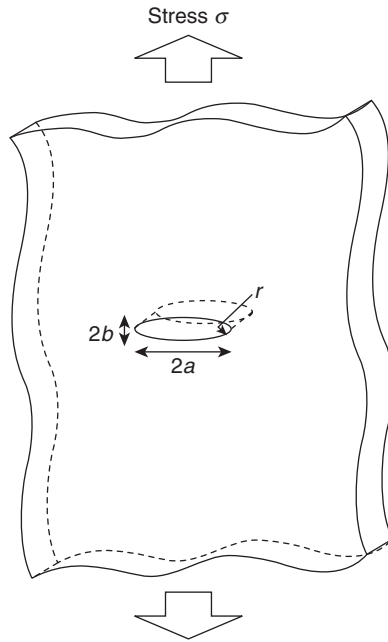


Figure 5.4 An elliptical flaw in a plate under stress.

because $2\sqrt{a/r}$ is much greater than one in this instance. The expression $2\sqrt{a/r}$ represents the stress concentration factor applied to the stress σ to give the local stress at the crack tip.

As the crack tip radius becomes smaller the stress at the crack tip becomes greater, so that a flat, sharp crack gives a very high stress concentration. This explains why sharp cracks have very high stresses at the crack tip. This is also the justification for sometimes drilling a hole at a crack tip in order to arrest its propagation, because the larger radius at the hole reduces the stress concentration factor at the crack tip and halts the fracture risk of that flaw. The hole-drilling technique can be difficult to apply because it is essential to ensure that the crack tip has been correctly located and drilled, and this may not be possible for the branched cracks found in stress corrosion cracking where there is no distinct crack tip. It also assumes that drilling a hole in the sheet is not intrinsically damaging to the component, such as causing leaks, etc.

5.6.2 Stress intensity factor K

Although the stress concentration factor is very useful in many engineering applications, the concept has two limitations. First, it describes only the stress at the precise position of the stress raiser and provides no information at all about the local stress just ahead of the stress concentration. Second, as the notch tip becomes increasingly sharp, i.e. the crack tip radius r becomes very small, the stress at the

notch tip given by equation 5.3 becomes very large. For a crack where we assume r approaches zero, the local stress and the stress concentration factor become very large and approach infinity. For dealing with cracks, therefore, we need a different approach.

The stress intensity factor K overcomes these two limitations of the stress concentration factor. A quite involved mathematical treatment of cracked bodies under stress is used to show that in an infinite plate the stress σ_x at any distance x ahead of the crack tip is given, to a good approximation, by:

$$\sigma_x = \frac{\sigma\sqrt{\pi a}}{\sqrt{2\pi x}} \quad (5.4)$$

where σ is the applied stress and a is the crack length.

The stress distribution ahead of the crack is shown in Figure 5.5.

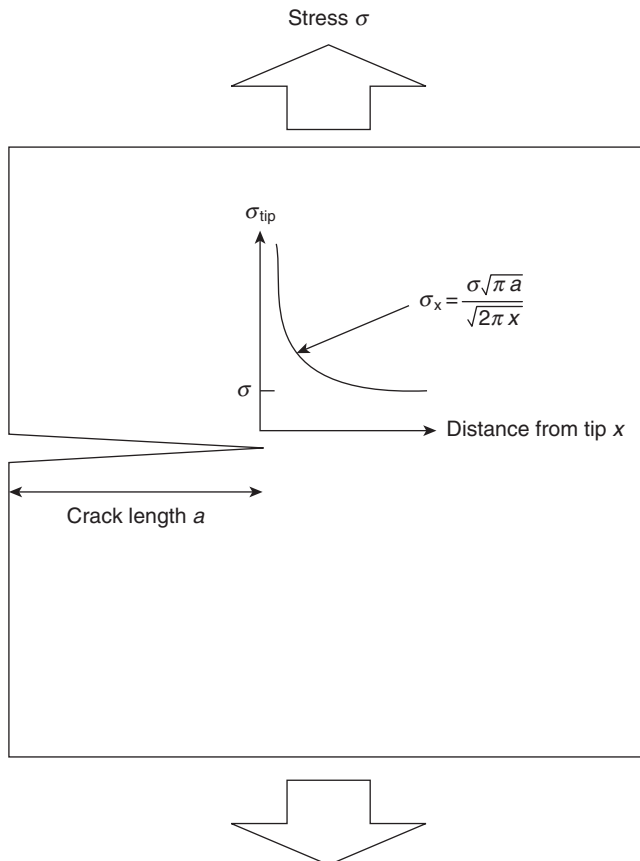


Figure 5.5 Stress distribution ahead of a crack tip.

The term $\sigma\sqrt{(\pi a)}$ is given the symbol K and is known as the stress intensity factor. Thus equation 5.4 can be rewritten as:

$$\sigma_x = \frac{K}{\sqrt{2\pi x}} \quad (5.5)$$

If the stress intensity factor is known, then the stress distribution ahead of the crack is then also known. Furthermore, if two cracks of different lengths are subjected to different applied stresses such that their associated stress intensity factors are the same, then the stress distributions ahead of both cracks are identical. It is reasonable to assume that the behaviour of a crack in a given material, i.e. whether or not it will extend by brittle fracture, depends only on the stress distribution ahead of the crack, which is, in turn, itself described completely by the stress intensity factor K . A single parameter K can therefore be used to describe crack behaviour.

In this simple treatment, the local stress at the crack tip approaches infinity. From a mathematical point of view, this is not a problem. As engineers, however, we know that infinitely high stresses cannot occur and that the material yields locally when the stress is greater than the yield strength. There is, therefore, a small region just ahead of the crack tip where yielding has occurred; this region is known as the plastic zone. Provided that the size of the plastic zone is small compared with the length of the crack and the component dimensions, the mathematical approach to the definition of stress intensity factor is still applicable; this approach is known as linear elastic fracture mechanics. If the size of the plastic zone becomes appreciable with respect to other dimensions, then the effects of this plastic zone must be taken into account and the approach used is elastic-plastic fracture mechanics (see 5.6.3 and 5.6.4).

The simple expression for the stress intensity factor

$$K = \sigma\sqrt{\pi a} \quad (5.6)$$

was initially derived for a specific basic case of a through-thickness crack in an infinite plate. A geometry factor Y is introduced to account for practical cases of other crack shapes and where the crack is located in a finite geometry. The most useful definition of the stress intensity factor is:

$$K = Y\sigma\sqrt{\pi a} \quad (5.7)$$

where Y depends on geometry, loading mode and crack shape and location.

Some examples of the influence of the geometry on the stress intensity factor are shown in Figure 5.6. For a through-thickness centre crack in a large plate the geometry factor Y is equal to one. The centre crack is defined as having a length of $2a$ because it has two crack tips. If this plate is cut in half, so the crack is now an edge crack with one crack tip and length a , then the open surface makes it easier to drive the crack open and hence the Y factor increases by 12% relative to the centre crack case and it therefore requires less stress to reach the same stress intensity level in this edge crack. Conversely, if the original defect of length $2a$ becomes a fully-embedded

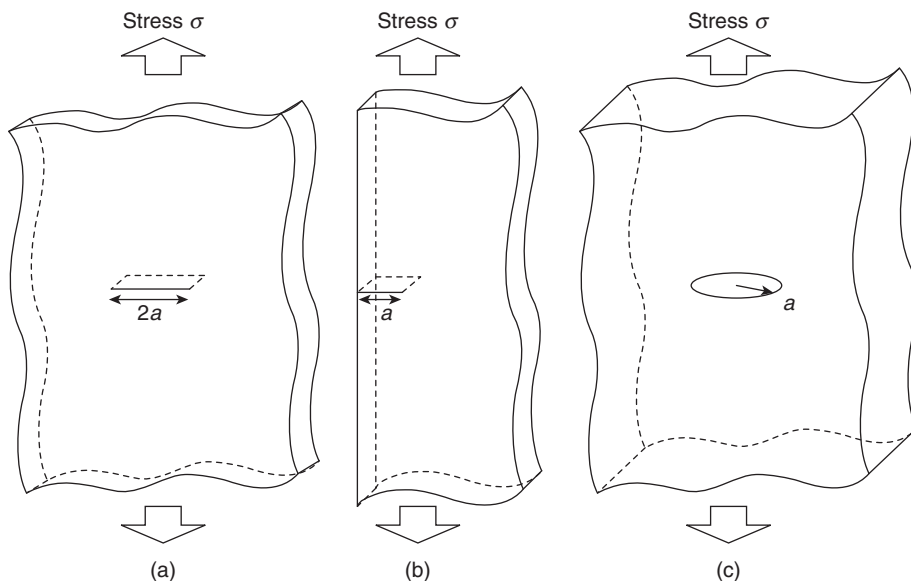


Figure 5.6 Geometrical factors for (a) centre crack ($Y = 1$), (b) edge crack ($Y = 1.12$), and (c) penny-shaped crack, ($Y = 0.637$).

penny-shaped crack rather than a through-thickness crack, then there is no open surface around the defect and the stress intensity factor reduces by 36%. It takes a greater applied stress to reach the equivalent stress intensity for this crack.

The difference in stress intensity factor between a centre crack and an edge crack is easily demonstrated with two pieces of paper. One piece of paper is folded in half lengthways and a small tear is made so that, when the paper is opened, there is a through-thickness crack in a large plate (equivalent to Figure 5.6a). On the second piece of paper, a small tear is made at the edge of the paper (equivalent to Figure 5.6b). Each piece of paper is then tested by holding it as evenly as possible across the top and bottom to tear it in half. It is much easier to rip the paper that is torn at the edge, because the stress intensity factor is higher at its crack tip and thus it requires less applied stress to propagate the crack.

The Y factor is readily found for simple geometries, but becomes more involved for other geometries. Expressions for Y are published in fracture mechanics handbooks and in BS 7910 for the most commonly encountered geometries. In addition, finite element modelling techniques can be used to calculate Y factors (sometimes described as stress intensity factor solutions) for specific cases.

The stress intensity factor formula can be used to calculate the stress field near the crack tip that is driving the crack towards unstable fracture. The stress intensity or crack driving force is counteracted by the material's fracture toughness (Figure 5.7). Once the applied stress intensity factor K is greater than the material's fracture toughness K_{mat} then the crack becomes unstable and brittle fracture occurs.

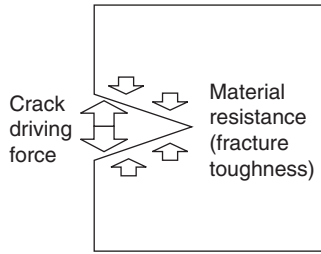


Figure 5.7 The balance between an applied stress causing a stress intensity factor which drives the crack to open, and the material's fracture toughness resisting the applied stress intensity.

Therefore, K can describe both the stress intensity factor and the material resistance that opposes it. The units of K are $\text{N mm}^{-3/2}$ or $\text{MPa } \sqrt{\text{m}}$ (or $\text{ksi } \sqrt{\text{in}}$ in the USA). These units can seem rather inelegant to those new to fracture mechanics because of the square root, but it is possible to convert between SI and US customary units using the following conversion factors:

$$1 \text{ N mm}^{-3/2} = 0.0316 \text{ MPa } \sqrt{\text{m}} \text{ (which is equal to } 1/\sqrt{1000})$$

$$1 \text{ N mm}^{-3/2} = 0.029 \text{ ksi } \sqrt{\text{in}}$$

5.6.3 Linear elastic fracture mechanics (LEFM)

The linear elastic fracture mechanics (LEFM) approach is based on the stress intensity factor alone. It assumes that materials under load behave essentially only in an elastic way, like the elastic part of the stress–strain graph where Hooke's law holds and there is a rectilinear relationship between stress and strain (Figure 5.8). Elastic stress–strain behaviour is described in section 4.3. This is known as the stress intensity factor approach and is a reasonably good model of the behaviour of cracks in brittle materials. However, this treatment assumes only a negligible amount of plastic deformation occurs at the crack tip and is not representative of what happens in reality for more ductile materials. Nevertheless, LEFM forms the basis for further modifications within fracture mechanics to allow for more extensive plastic deformation at the crack tip.

5.6.4 Elastic–plastic fracture mechanics (EPFM)

In ductile materials, the LEFM approach is no longer adequate to describe the behaviour of cracks, and modifications to the fracture mechanics solutions are made to account for the plastic deformation at the crack tip. This approach is known as elastic–plastic fracture mechanics (EPFM). The ductility of the material gives rise to a relatively large plastic zone at the tip of the crack. Within the plastic zone, the material locally yields and deforms, thus absorbing energy and, hence, increasing

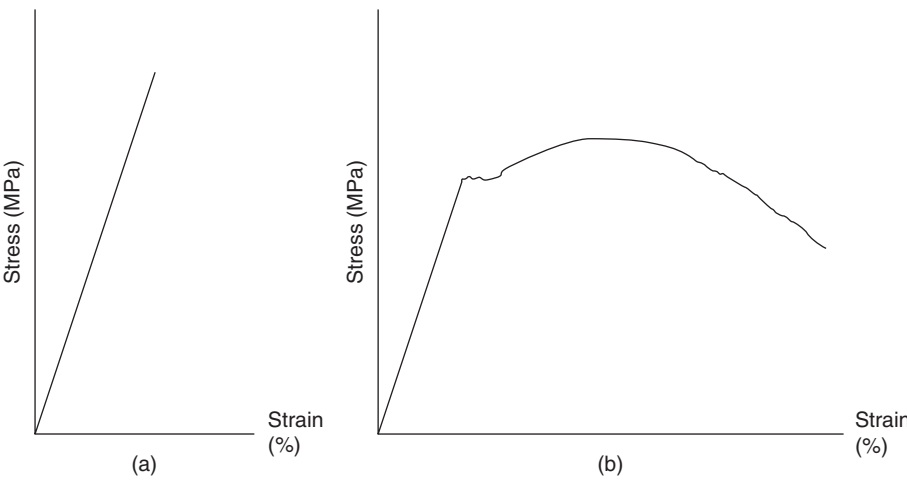


Figure 5.8 Stress–strain curves for (a) a linear elastic material which shows no ductility and (b) a more ductile steel whose behaviour is described by elastic–plastic fracture mechanics.

Table 5.1 Comparison of LEFM, as illustrated in Figure 5.9, and EPFM, as illustrated in Figure 5.10, showing the key differences between the two fracture mechanics approaches

LEFM	EPFM
Linear elastic fracture mechanics	Elastic plastic fracture mechanics
Brittle materials	Ductile materials
Negligible plastic zone at crack tip	Large plastic zone at crack tip
Crack tip stress assumed to be very high	Crack tip stress limited to yield strength
Low fracture toughness	High fracture toughness

toughness. The required applied stress to cause unstable fracture is much larger than that anticipated by LEFM. As the size of the plastic zone increases, there is more energy absorbed and the material exhibits a higher fracture toughness, and more ductility. The differences between LEFM and EPFM are summarised in Table 5.1 and Figures 5.9 and 5.10.

5.6.5 Ductility, thickness and the crack tip plastic zone

The ductility of an engineering material depends on a number of factors. The plastic deformation associated with ductility at an atomic scale depends on the movement of atoms in relation to one another. Under a high strain rate loading, there may be

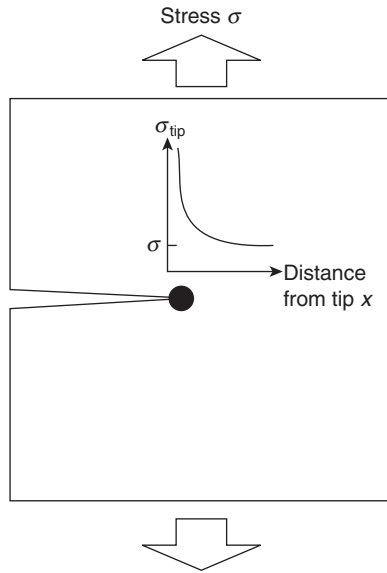


Figure 5.9 LEFM model of the crack tip, showing a small plastic zone size and a plot of crack tip stress σ_{tip} versus distance x from the crack tip.

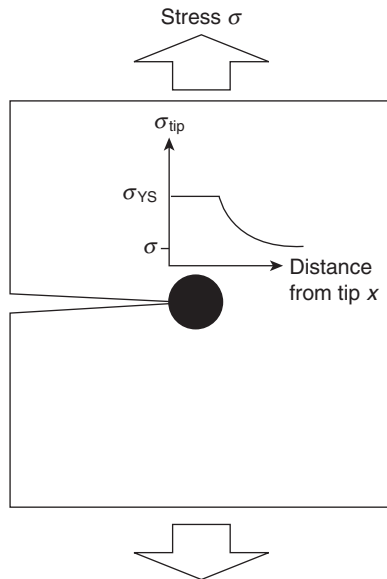


Figure 5.10 EPFM model of the crack tip, showing a large plastic zone size and a plot of crack tip stress σ_{tip} versus distance x from the crack tip.

insufficient time for this deformation to occur and the material therefore behaves in a less ductile way than if loaded under a slower strain rate. A similar effect is seen at low temperature when the thermal energy available to allow atomic motion is reduced so that, compared with the equivalent material at higher temperatures the material is less ductile. These effects of high strain rate and low temperature reducing ductility affect the bulk material's tensile behaviour, i.e. alter the effective stress–strain curve.

The extent of the plastic zone (z_p) is related to the applied stress intensity factor K and the yield strength σ_{YS} as given in equation 5.8.

$$z_p \sim \left(\frac{K_I}{\sigma_{YS}} \right)^2 \quad (5.8)$$

One influence on the plastic zone size is the local material yield strength at the crack tip, and this, in turn, depends on the microstructure. This is particularly the case for the weld heat affected zone (HAZ), where hard brittle microstructures such as martensite can form close to the fusion line in steels with moderate carbon levels. Alternatively, particularly coarse grains within the HAZ may also act as local brittle zones that offer little resistance to crack propagation. In some joints, the structural integrity of the whole weld is limited by the behaviour of cracks located in these small brittle regions, independent of the bulk material structural behaviour.

Another influence on the plastic zone size is material thickness, owing to a characteristic known as constraint. For a material loaded under axial tension (Figure 5.11a), as it yields it tends to elongate. However, during this elongation, there are also associated compressive strains acting perpendicularly to the direction of tension owing to Poisson's contraction (Figure 5.11b). When a material is restrained, it cannot

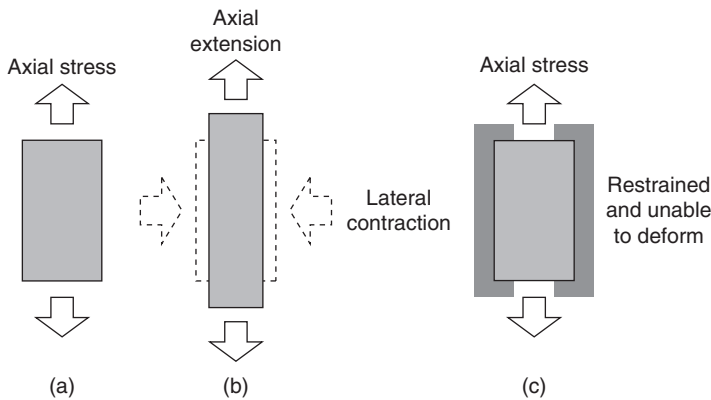


Figure 5.11 (a) An axial load is applied to a piece of material causing (b) axial extension and lateral contraction owing to Poisson's contraction (c) if a material is restrained it does not deform in this way under load.

deform in the same way as under free conditions. These are known as constraint conditions.

In a thick plate, at the outer surfaces, the material is able to deform laterally in a similar way to the free material in Figure 5.11b and this is associated with some ductility. However, towards the middle of the plate, the material is constrained by the surrounding material and it experiences stresses as shown in Figure 5.11c. In this situation, ductility is reduced and the plastic zone size is smaller.

The stress condition where lateral contraction occurs is called 'plane stress'. Plane stress conditions occur in thin materials and also near the surfaces of a thick plate. The constrained condition where lateral contraction is not possible is called 'plane strain', and this occurs at the centre of a thick plate.

The effect of thickness on a crack's behaviour may be visualized by imagining an edge crack in a thick plate (Figure 5.12). Near the plate surfaces under plane stress conditions, the plastic zone size at the tip of the crack is relatively large, but the plane stress conditions only extend a certain distance from the plate surface. Between these regions where plane strain conditions exist, the plastic zone size is reduced by a factor of three and, here, the material effectively behaves in a much less ductile way.

In thin plate, the plane strain part of the plastic zone is proportionately much less than the plane stress part, and the plane stress conditions dominate the performance of the crack (Figure 5.13). In very thin material, the plastic zones for each of the surfaces overlap in the middle of the plate and the whole material experiences only plane stress conditions. However, in thick material the proportion of plane strain conditions along the middle of the crack front governs the behaviour of the whole structure. Structural steels greater than around an inch thick (25 mm) are sufficiently thick to generate significant amounts of plane strain conditions, and therefore have a higher risk of brittle fracture. Steels less than 10 mm thick usually behave in a ductile way. This is why a thick plate has a lower fracture toughness than a thinner plate of otherwise identical material.

5.7 Fracture toughness parameters

There are three different fracture parameters that may be used to describe both the crack driving force and the fracture toughness: the stress intensity factor K , the crack tip opening displacement (CTOD) δ , and the J integral. The most suitable fracture parameter depends on the toughness behaviour of the material: whether it is ductile, brittle or can exhibit a range of toughness properties.

The first fracture parameter introduced in this chapter was the stress intensity factor K , a stress-based parameter that was used to describe the basic concepts of linear elastic fracture mechanics. K is the most appropriate fracture parameter for brittle materials because it does not take account of any significant ductility. Often the material's fracture toughness is expressed as K_{mat} , whereas the applied stress intensity is K_I (where the subscript I refers to the crack opening mode). There are clearly defined standard test methods for defining K_{mat} and, when the conditions for linear elastic conditions are fully met, K_{mat} is known as K_{Ic} .

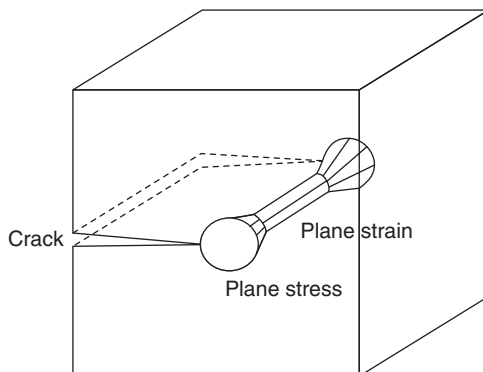


Figure 5.12 Illustration of the relative plastic zone sizes under plane stress and plane strain conditions, near the plate surface, and in the centre of a thick plate respectively.

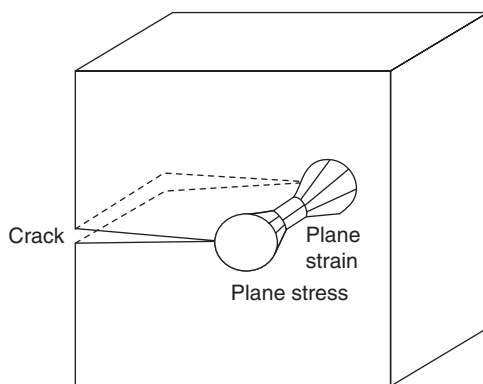


Figure 5.13 Illustration of the plastic zone sizes under plane stress and plane strain conditions, the relative amounts of the plane stress and plane strain conditions for a thinner material (compared with Figure 5.12).

For ductile materials that show upper-shelf behaviour, it is more appropriate to use the J integral, or simply J , which is an energy-based parameter. The fracture mechanics description of J is related to the conversion of elastic strain energy into new crack surface and plastic deformation as the crack extends. The units of J are J m^{-2} .

The third option is the crack tip opening displacement (CTOD), which is a strain-based parameter also known as δ , representing the degree of opening at the crack tip owing to plastic deformation. As the load increases, CTOD increases until the crack starts to extend at a critical value of CTOD, which defines the material fracture toughness δ_{mat} . The units of CTOD are mm as it is simply a measurement of length. It is important to recognise that fracture toughness when measured using CTOD is not an absolute material property, but depends on material thickness as described in

the previous section. For engineering purposes, therefore, CTOD toughness relates only to a specific material of a specific thickness at a given temperature. CTOD is perhaps the most versatile fracture parameter because it can be used to describe both ductile and brittle materials.

It is possible to determine the fracture toughness as expressed by any or all of these parameters from the same fracture toughness test, based on the load–displacement plot and dimensional measurements from the test specimen. Equations to calculate K , CTOD and J are given in the fracture toughness testing standards. Fracture toughness testing is discussed in more detail in Section 9.6. Certain industries and countries have their own preferences for describing fracture toughness. K is often used by the UK nuclear industry where the concern is risk of brittle fracture after irradiation. CTOD, developed in the UK, is commonly used there and also tends to be preferred by the pipeline and offshore industry. In the USA, it is sometimes more common to use J rather than CTOD, because J was originally developed in the USA.

5.8 Conclusions

Brittle fracture is probably the most catastrophic failure mode in welded structures. The risks of brittle fracture can be understood in general terms and avoided if fabrication is to an established code. The reasons why low temperature, high thickness, high residual stress and the presence of defects all contribute to the likelihood of brittle fracture can be explained by the principles of fracture mechanics, which describes the behaviour of cracks in structures.

By understanding the application of fracture mechanics, it is possible to perform fracture mechanics (fracture toughness) tests (described in Section 9.6), and to assess the significance of defects using fracture mechanics based assessment procedures (discussed further in Chapter 11).

5.9 Sources of further information and advice

- ABS (2014) *Rules for building and classing steel vessel*, American Bureau of Shipping.
- Anderson, T. L. (2005) *Fracture mechanics: fundamentals and applications, third edition*, CRC Press; ISBN-10: 0849316561, ISBN-13: 978-0849316562.
- BS EN 1990 (2002). *Eurocode, basis of structural design*, British Standards Institution.
- BS EN 1993-1-1 (2005). *Eurocode 3. Design of steel structures – general rules and rules for buildings*, British Standards Institution.
- BS 4515-1 (2009) *Specification for welding of steel pipelines on land and offshore – Carbon and carbon manganese steel pipelines*, British Standards Institution.
- BS PD 5500 (2012). *Specification for unfired fusion welded pressure vessels*, British Standards Institution.
- BS 7910 (2013). *Guide to methods for assessing the acceptability of flaws in metallic structures*, British Standards Institution.

- François, D., Pineau, A. and Zaoui, A. (2012) *Mechanical behaviour of materials: volume II: fracture mechanics and damage*, Springer, ISBN-10: 9400749295, ISBN-13: 978-9400749290.
- Gdoutos, E. E. (2010) *Fracture mechanics: an introduction (solid mechanics and its applications) 2nd edition*, Springer, ISBN-10: 9048167345, ISBN-13: 978-9048167340.
- Hertzberg, R. W., Vinci, R. P. and Hertzberg, J. L. (2012) *Deformation and fracture mechanics of engineering materials, 5th edition*, John Wiley & Sons; ISBN-10: 0470527803, ISBN-13: 978-0470527801.
- Janssen, M., Zuidema, J. and Wanhill, R. (2004) *Fracture mechanics, second edition*, CRC Press, ISBN-10: 0415346223, ISBN-13: 978-0415346221.

6.1 Introduction

In previous chapters, we have considered cases where the load is applied to the structure essentially only once, i.e. the permanent load as discussed in Chapter 2. In practice, however, many structures have to withstand many repeated applications of load, the variable loads also discussed in Chapter 2. The repeated application of a load is generally known as cyclic loading.

From the early days of the industrial revolution in the first half of the 19th century, structures were subjected to repeated applications of load. Pistons in steam engines underwent cyclic loading owing to the steam cycle pushing the piston backwards and forwards. These pistons then generated rotational movement in, e.g., mine shaft hoists or railway axles. These hoists and axles were subjected to bending loading and, each time the axle rotated, the stress at a point on the surface ranged from zero to tension to zero to compression and back to zero again (Figure 6.1).

Engineers of the time were well able to calculate these stresses and ensured that the stresses were well below the yield strength of the iron or steel, the established good practice of the day. Nevertheless, mine hoists and railway axles failed, and perhaps the most significant was the railway accident that occurred just outside Paris at Versailles in 1842. More than 50 people lost their lives and it was subsequently established that the cause of the accident was a broken locomotive axle. A sketch of the broken axle is shown in Figure 6.2. Although not widely recognised at the time, the failure exhibited two characteristics that are typical of failures under cyclic loading:

- a) the fracture location was at a stress raiser, in this instance the notch where there was a small change in axle diameter; and
- b) the fracture surface was flat, relatively featureless and at right angles to the applied stress.

To 19th century engineers, these fractures were puzzling as it was clear that even cyclic loads well below the elastic limit were sufficient to cause failure. The notion developed that the repeated application of load somehow caused the iron or steel to become ‘tired’. It may be that this notion was a direct parallel with a worker becoming tired after a day’s toil of repetitive tasks. As a result, failure under cyclic loading became known as fatigue failure, and application of repeated loads became known as fatigue loading.

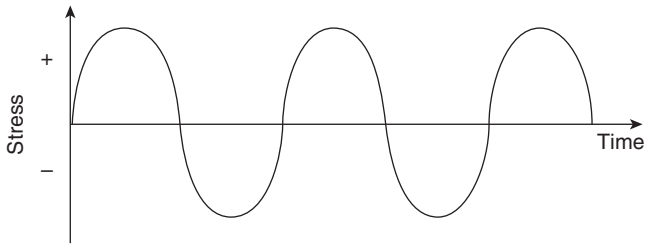


Figure 6.1 Stress cycling from tension (+) to compression (–) as a result of axle rotation.

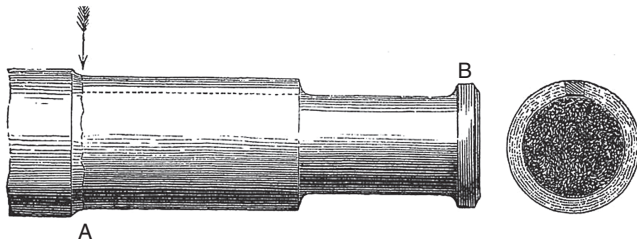


Figure 6.2 Fatigue failure in a train axle after the Versailles train crash (by Joseph Glynn, 1843).

We now know that the notion of the material becoming ‘tired’ is false; the fracture is caused by the initiation and steady growth of cracks, but the terminology ‘fatigue’ is now firmly entrenched.

The study of the behaviour of materials under cyclic loading started in the mid-19th century as a first step towards preventing fatigue failures. At this time, welding had not been developed as a fabrication route and the investigations were mainly concerned with unwelded, or plain, material. As an understanding of how plain materials fail by fatigue is essential, in this chapter the principles of fatigue are discussed in relation to the performance of unwelded material. Chapter 7 demonstrates the dramatic (and negative) influence that welding has on fatigue performance.

6.2 Engineering perspective of fatigue

The periphery of a railway axle undergoes a very simple stress cycle as the axle rotates. A simple fatigue loading cycle is shown in Figure 6.3, and the following terms are defined:

- maximum stress: the largest stress in the cycle;
- minimum stress: the smallest stress in the cycle;
- mean stress: the average of the minimum and maximum stresses;
- stress range: the algebraic difference between the maximum and the minimum stress;

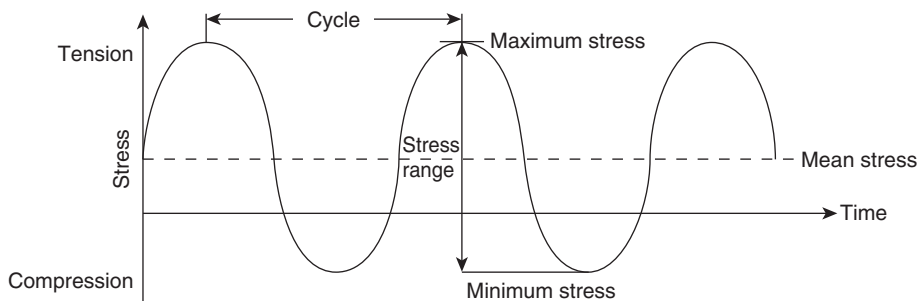


Figure 6.3 Definitions of stress in cyclic loading.

cycle: the interval between one stress peak and the next; and
frequency: the number of cycles per second.

In most design approaches the stress range is used as the important parameter controlling performance. The stress cycle in Figure 6.3 is relatively simple in that the maximum and minimum stresses do not change from one cycle to the next. This is what we would expect the loading to be on a railway axle, and it is known as constant amplitude loading.

In structures such as bridges and offshore platforms, the maximum and minimum stresses depend on, for example, the mass of the vehicle crossing the bridge, or the size of the wave impacting the platform. In these circumstances, the stress history is complex and is known as variable amplitude loading. The method of treating variable amplitude loading in a fatigue assessment is discussed further in Chapter 7.

The most natural way to study fatigue behaviour is to take a piece of material, apply a cyclic load such as shown in Figure 6.3 and simply count the number of cycles that it takes to cause the material to break. By carrying out a series of tests to failure on different specimens at different stress ranges the results can be plotted as shown in Figure 6.4.

This type of diagram is called an S–N diagram, from the stress range (S) and the number of cycles to failure (N). Figure 6.4 shows schematically the results of several tests on similar specimens. Tests at high stress ranges correspond to a small number of cycles to failure (or endurance); at lower stress ranges, the endurance increases. The best fit line between the points is called the S–N curve. Note that the experimental results exhibit scatter and do not all sit exactly on the line. This scatter in results is characteristic of fatigue experiments.

As the stress range is reduced, the corresponding fatigue endurance increases and the S–N curve becomes parallel to the endurance (N) axis. At this stage, the fatigue life is very long and for practical purposes we can define an endurance or fatigue limit, which is the stress range below which it can be assumed that fatigue failure does not occur.

For many materials subject to cyclic loading in air, the position of the S–N curve does not depend for practical purposes on the frequency of the loading; i.e. the number

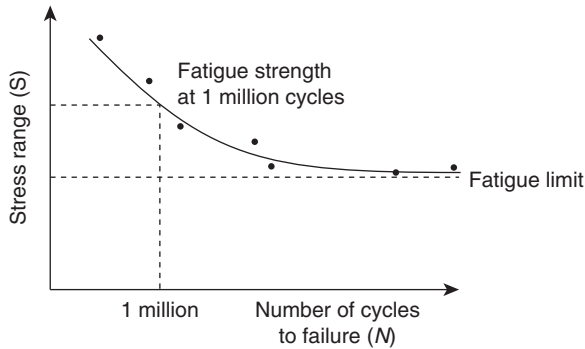


Figure 6.4 Stress versus the number of cycles to failure plotted for a number of identical specimens tested under different stress ranges.

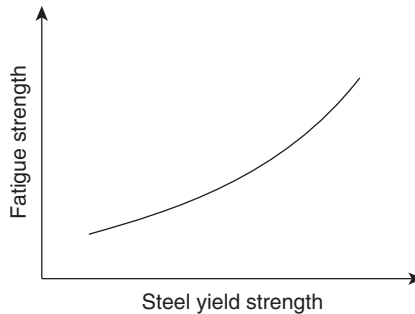


Figure 6.5 Effect of yield strength on fatigue strength of steel.

of cycles to failure is not influenced by how rapidly the cycles are applied. For example, if the endurance of a material was 1 million cycles, failure would occur after 116 days if the frequency was 1 cycle per second (1 Hz), or after 27 000 years if the frequency was 1 cycle per day. It is always essential to be clear when using terms such as 'lifetime', which generally refers to the number of cycles rather than an elapsed period of time.

To compare the performance of different materials, it is convenient to define a fatigue strength. This is the stress range corresponding to failure after a given number of cycles. One million cycles is often taken as the benchmark and Figure 6.4 shows the fatigue strength at one million cycles. Other endurances could also be chosen and fatigue strength at two million cycles is also commonly used.

Using the concept of fatigue strength, it is simple to illustrate the importance of major parameters. Figure 6.5 shows that, for steel, as the material yield strength is increased (by, for example, heat treatment) fatigue strength also increases. In addition, increasing the surface roughness of the material reduces fatigue strength (Figure 6.6). One simple way of improving fatigue performance is therefore to polish the surface smooth. Rough surfaces can be microscopically shown to consist of small hills and

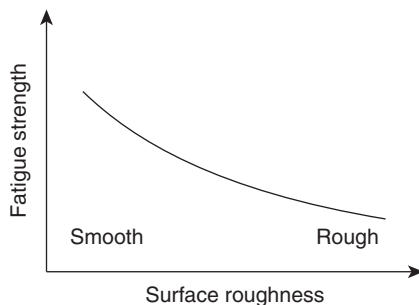


Figure 6.6 Effect of surface roughness on fatigue strength of steel.



Figure 6.7 Typical (a) blunt and (b) sharp notches in fatigue specimens to demonstrate the effect of notches on fatigue strength (see Figure 6.8).

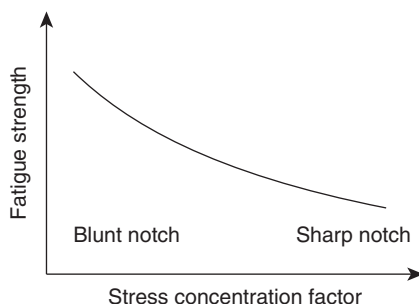


Figure 6.8 Influence of notches on fatigue strength.

valleys, with the valleys looking very similar to small surface notches. This can be taken a stage further by deliberately introducing notches in the otherwise smooth fatigue specimen, as shown in Figure 6.7. Each of these notches acts as a stress raiser and the stress at the notch root is magnified by a stress concentration factor, depending on the detailed notch geometry. The sharper the notch, the higher the stress concentration factor. Fatigue strength decreases with increasing stress concentration, as shown in Figure 6.8.

In Figure 6.1, the mean stress is zero, which would be the case for a rotary shaft. In other circumstances, however, such as when the cyclic loading is imposed on a permanent load, the mean stress may not be zero, as defined in Figure 6.3. The stress ratio (R), defined as the minimum stress divided by the maximum stress, is a convenient

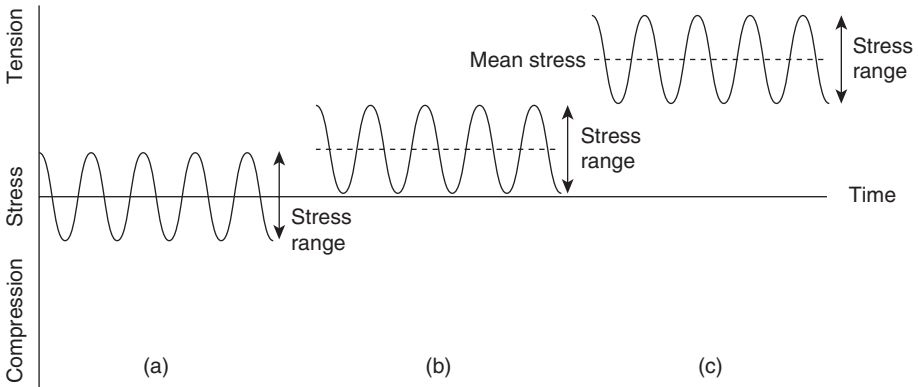


Figure 6.9 Effect of mean stress on fatigue strength: (a) zero mean stress (alternating loading), stress ratio = -1 , (b) zero minimum stress (pulsating loading), stress ratio = 0 , (c) tensile minimum stress (wholly tensile loading), stress ratio > 0 .

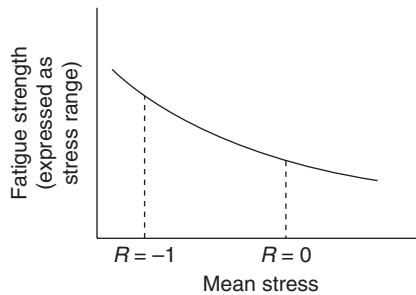


Figure 6.10 Fatigue strength decreases as the mean stress increases.

way of defining the mean stress, as shown in Figure 6.9. In Figure 6.9, each loading cycle has the same stress range, but a different mean stress. When R is negative, at least part of the stress cycle is in compression. When R is positive (or zero), the entire stress cycle is tensile. The influence of mean stress on the fatigue strength is shown in Figure 6.10.

In Figure 6.9, the fatigue strength is expressed in terms of the total stress range, increasing the mean stress reduces the fatigue strength until $R = 0$, i.e. the whole stress cycle is tensile. When the stress cycle is partly compressive, the stress range does not appear to be as damaging as when the whole stress cycle is tensile.

6.3 Metallurgical perspective of fatigue

As mentioned in 6.1, it was initially thought that cyclic loading caused some undetectable changes in the material that caused it to become 'tired' or 'fatigued'.

We know that failure under cyclic loadings occurs as a result of the initiation and steady growth of a crack across the load-bearing area. Because the crack grows relatively slowly and stably in most cases, fatigue crack growth is known as a subcritical crack extension mechanism.

Fatigue crack initiation occurs in material grains that by chance have their crystallographic planes oriented particularly favourably with respect to the applied direction of cyclic stressing. This orientation is such that crystal slip planes are at 45° to the applied stress and, hence, a very small amount of dislocation movement occurs in these grains, even though the applied stress is well below yield. Dislocations move in one direction when a tensile load is applied and in the opposite direction when a compressive load is applied. These dislocations move on different planes and the net result is irreversible plastic flow that gives rise to small intrusions and extrusions on the material's surface. Continued cyclic loading causes this incipient crack to continue to grow at 45° to the applied stress range.

Such growth continues until the crack encounters a grain boundary, where the next grain almost certainly does not have the same favourable orientation. The growth of the crack across the first grain is often termed the fatigue initiation stage. At this point, if the conditions are favourable for continued crack propagation, the crack changes its direction of growth and now grows at right angles to the applied direction to stress. Each fatigue cycle, however, causes the crack to extend by only a very small amount. Fatigue crack initiation and propagation are shown in Figure 6.11. The total

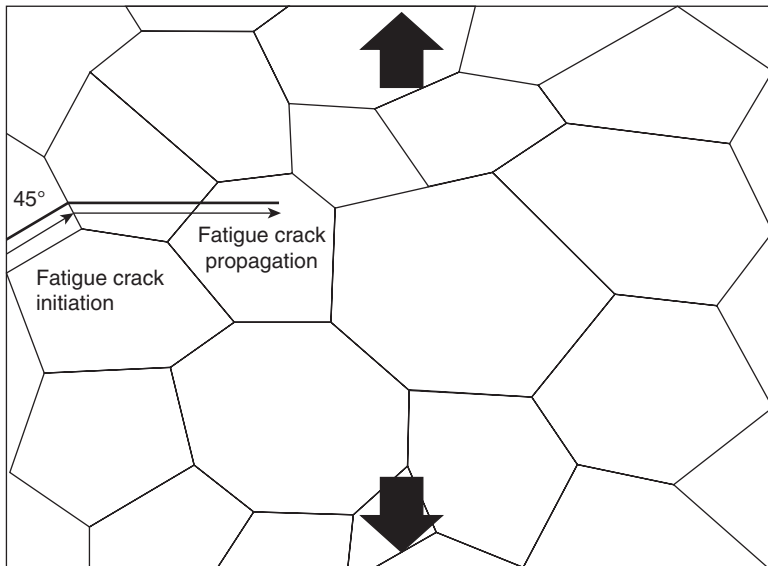


Figure 6.11 Fatigue crack initiation and propagation across the first few metal grains in a structure under cycling loading.

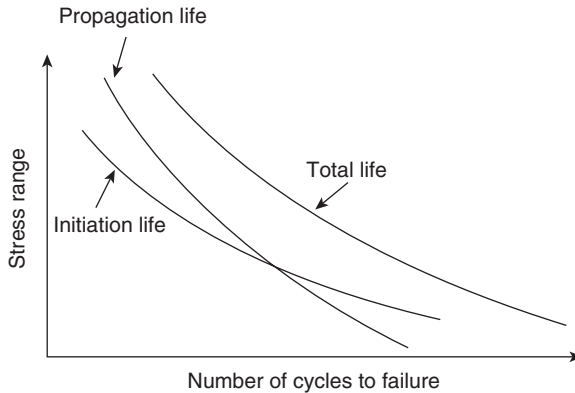


Figure 6.12 Total fatigue endurance shown as an S–N curve for the initiation and propagation stages.

fatigue endurance is therefore the sum of the cycles required for initiation plus the cycles required for propagation, and the S–N curve is shown in Figure 6.12.

Fatigue crack initiation becomes relatively more difficult at low stress and it is generally thought that the existence of a fatigue limit is the result of the stresses being so low that the extremely small amount of dislocation movement induced is insufficient to cause crack initiation.

All the observations that we considered from an engineering perspective in 6.2 can be explained by this simple microstructural model of fatigue crack initiation and propagation. As the material yield strength is increased, it becomes more difficult to induce dislocation movement and, hence, the crack initiation phase requires an increased number of cycles. Consequently, the material's fatigue strength is increased. Material with a rough surface finish is equivalent to material with small surface notches that locally increase the stress such that the initiation phase is completely absent. In a similar manner, the presence of a notch or stress raiser may cause the initiation phase to be absent and, therefore, the fatigue endurance to be reduced.

6.4 Practical implications for a growing fatigue crack

The metallurgical description of fatigue given in 6.3 is an accurate, if simplified, portrayal of the scientific mechanism of fatigue failure. The phenomenon of crack initiation at 45° to the applied stress and the subsequent propagation at 90° is only observed in carefully controlled laboratory tests and is almost never seen in real life structures. In actual structures, there is always some form of notch or stress raiser or rough surface that completely overrides any classical microstructural initiation phase. The fatigue crack starts to propagate from the stress raiser or notch immediately at right angles to the applied stress. In Chapters 7 and 12 we consider the fatigue crack propagation stage alone.

There is, however, a confusing use of the term ‘crack initiation’. It is often used by engineers to define the stage where the fatigue crack has extended to a particular size, say 1 mm in length, which can be detected by certain NDT methods (as described in Chapter 10). This is a completely different use of the expression ‘crack initiation’ from that used by classical metallurgists.

6.5 Conclusions

Fatigue is a failure mechanism that occurs by the repeated application of stresses; the single application of this stress being insufficient to cause failure. Failure occurs by the slow and stable growth of a crack across the load-bearing area. The most common method of describing fatigue behaviour is through an S–N curve, which is a graph of applied stress range (S) against the number of cycles to failure (N).

The significance of fatigue for welds under cyclic loading is discussed in Chapter 7. The assessment of fatigue is described in Chapter 12, whereas methods of improving the fatigue performance of welds are covered in Chapter 13.

6.6 Sources of further information and advice

- Frost, N. E. (2003) *Metal fatigue* Dover Publications Inc., ISBN-10: 0486409279, ISBN-13: 978-0486409276.
- Lewis, P. R. and Nisbet, A. (2008) *Wheels to disaster!: The Oxford train wreck of Christmas Eve 1874*, The History Press, ISBN-10: 075244512X, ISBN-13: 978-0752445120.
- McEvily, A. J. (2001) *Metal failures: mechanisms, analysis, prevention*, John Wiley & Sons, ISBN-10: 0471414360, ISBN-13: 978-0471414360.
- Milella, P. P. (2012) *Fatigue and corrosion in metals*, Springer, ISBN-10: 8847023351, ISBN-13: 978-8847023352.
- Stephens, R. I., Fatemi, A., Stephens, R. R. and Fuchs, H. O. (2000), *Metal fatigue in engineering, 2nd edition*, John Wiley & Sons, ISBN-10: 0471510599, ISBN-13: 978-0471510598.

7.1 Introduction

In Chapter 6, we introduced the concept of cyclic loading, also known as fatigue loading, and showed that under these circumstances failure occurs by the initiation and subsequent steady growth of a crack across the load-bearing member. We emphasised, however, that the treatment of fatigue in Chapter 6 concerned only plain, unwelded material. The fatigue behaviour of welded steels differs from that of unwelded material in many aspects. In this chapter, these differences are described and explained. Despite what may seem like a succession of depressing facts, we show how a clear, simple, and reliable methodology for fatigue design of welded joints has been developed.

7.2 Fatigue performance of welded joints

In the middle of the twentieth century, extensive new motorway building programmes were undertaken, particularly in the USA, Europe and Japan. These new motorways inevitably involved the fabrication of many bridges which were constructed as welded steel fabrications. The engineers recognised that these bridges had to be designed to withstand fatigue loading arising from the passage of vehicles. At the time, there was little guidance available on the fatigue design of welded joints and, to remedy this lack of knowledge, large experimental programmes of fatigue tests were carried out. These programmes and subsequent research work provided the basis for our current knowledge of the fatigue behaviour of welded joints. We examine several of the vital features to recognise in the following subsections.

7.2.1 *Poor fatigue performance*

Figure 7.1 shows schematically a typical S–N curve for an unwelded specimen made from a structural steel and also an S–N curve for a similar specimen made from the same steel, but this time with an attachment welded to one side. The results from both the unwelded and welded material exhibit scatter, which is entirely normal in fatigue data, as discussed in Chapter 6.

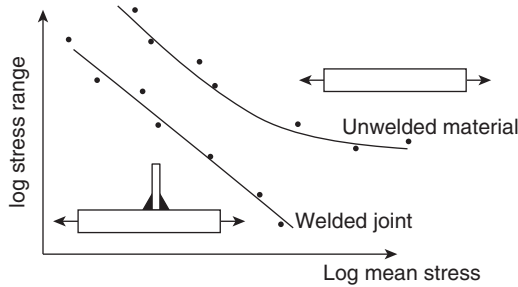


Figure 7.1 Comparison of the S–N curves for structural steel specimens, unwelded and with a welded attachment.

It is clear that the fatigue behaviour of the welded joint is dramatically poorer than that of the unwelded material. The two specimens are identical except that one has a seemingly ‘harmless’ attachment welded onto its surface. However, this attachment is not ‘harmless’ at all from a fatigue point of view; its presence results in a much inferior performance.

Fatigue data from unwelded material indicate the existence of a fatigue limit, i.e. the stress below which fatigue failure does not occur. In contrast, test results from welded joints show no indication of a fatigue limit; even relatively low applied stress ranges eventually give rise to fatigue failure in welds, although the associated number of cycles to failure is large. Research is continuing into whether welded joints do eventually exhibit a fatigue limit or not. The main point to emphasise however, is that the fatigue performance of welded joints is very poor compared with plain unwelded material.

7.2.2 No influence of steel yield strength

It has long been recognised that for unwelded material, fatigue strength increases with the material yield strength. When the first fatigue tests were carried out on similar joints welded from steel grades of increasing yield strength it was fully expected that the welded joints would show the same behaviour. It was a great surprise when the experimental results showed that the fatigue performance of a joint made from steel with a yield strength of 250 N/mm^2 was the same as that for a similar joint made from steel with a yield strength of 750 N/mm^2 ! This is shown schematically in Figure 7.2; the fatigue strength of a welded steel joint does not depend on the yield (or ultimate) strength of the steel.

7.2.3 No influence of applied mean stress

In unwelded material, an increase in mean stress generally results in a decrease in fatigue strength, particularly when part of the stress cycle is compressive. Under compressive stress, the fatigue crack does not fully open and, in this part of the stress cycle, the fatigue crack does not propagate. However, for welded joints the fatigue

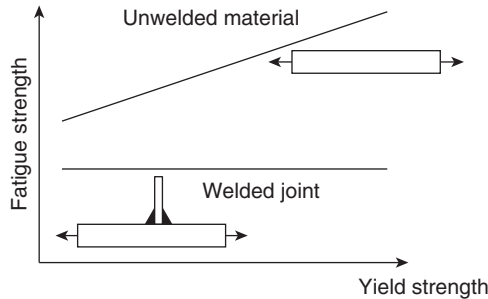


Figure 7.2 Relative fatigue performance of welded and unwelded joints, made from a range of steel with different yield strengths.

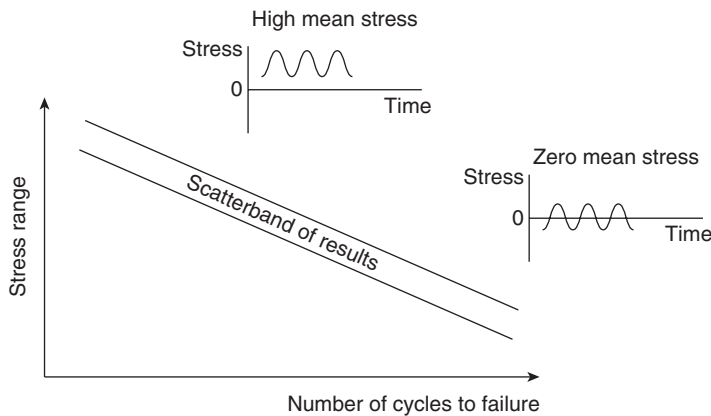


Figure 7.3 Fatigue performance is independent of applied mean stress, shown as an S–N curve based on total stress range.

performance, expressed in terms of the number of cycles at a given stress range, is independent of applied mean stress. This is a consequence of the high residual tensile stress levels induced in the joint area by the welding operation. Figure 7.3 shows schematically the independence of the S–N curve to the applied mean stress.

7.3 Special features of welded joints

In Section 7.2, we explained that the fatigue performance of welded joints was very poor, that fatigue strength did not depend on material yield strength, and that the applied mean strength did not influence fatigue behaviour. There are several special features associated with welded joints that are responsible for their very poor fatigue strength.

7.3.1 Stress concentration

Welded joints inevitably introduce a geometrical disruption in a structure, giving rise to a stress concentration, which is a small localised region where the stress is higher than the applied nominal stress. Even a thickness change can give rise to local regions of higher stress at the thickness transition and any kind of notch acts as a significant stress concentration (see 5.2). This stress concentration effect can be seen even in a simple weld bead on plate, Figure 7.4, where the weld toe corresponds to a notch and introduces a stress concentration.

The geometry of a fully penetrating butt weld is similar to the bead-on-plate weld, except that now there are four positions where a stress concentration is introduced: the two weld toes at the weld face, and the two weld toes at the weld root, Figure 7.5.

The results of laboratory fatigue tests show that the fatigue crack starts to grow from the stress concentration and propagates across the load-bearing area. This is shown in Figure 7.6, which is a section through a cracked butt weld. Fillet welded attachments give rise to stress concentrations at the weld toes as some load is always transferred to the attachment, Figure 7.7. Once again, it is the introduction of a stress raiser that is responsible for the poor performance. A typical fatigue failure from the toe of a fillet weld is shown in Figure 7.8.

For joints that are essentially at right angles to the applied load, such as those in Figure 7.4 to 7.8, stress concentrations are generated at the weld toes and the weld root (particularly in partial penetrating welds). For welds that are parallel to the applied load, such as web to flange joints or longitudinal stiffeners, stress concentrations are generated at the ends of stiffeners, cope holes and, to a lesser extent, at weld ripples and stop-start positions. A typical example of a fatigue crack at the end of a stiffener is shown in Figure 7.9.

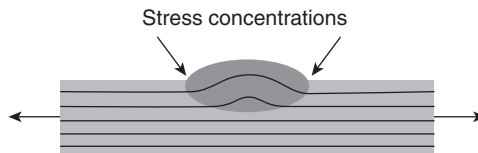


Figure 7.4 Stress concentration at weld toes of a weld bead on a plate, shown as the distortion of the lines of stress 'flow' through the structure.

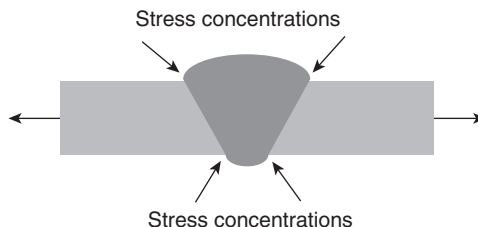


Figure 7.5 Stress concentrations at the weld cap and root toes in a butt weld.

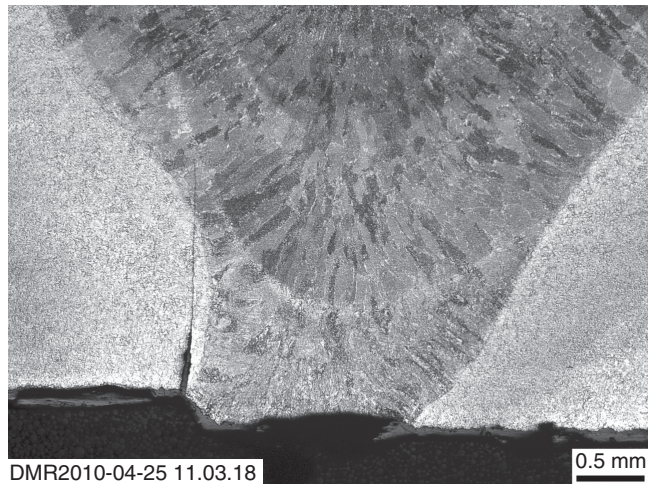


Figure 7.6 Section through a butt weld with a fatigue crack initiating at the toe of the weld root (© TWI).

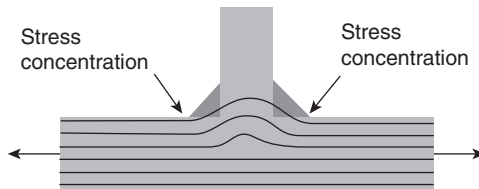


Figure 7.7 Stress concentrations in welded attachments

7.3.2 Weld toe intrusions

At the toes of welds are small intrusions which act as tiny microcracks, about $50\mu\text{m}$ deep. There is some evidence to suggest that these intrusions may comprise tiny non-metallic inclusions, trapped at the weld toe as a consequence of the welding process. These particles are microscopically smaller than the non-metallic inclusions distributed throughout the weld metal microstructure or along the segregation centreline of rolled-steel plate. The weld toe intrusions further magnify the stress concentration at the weld toes and behave like small pre-existing cracks, making it relatively easy to initiate a fatigue crack at these locations. The traditional view that a relatively large number of cycles may be required to initiate a crack in unwelded material no longer holds for welded joints. The combined effect of the stress concentration and the small intrusions causes the crack initiation phase to be almost completely absent, and the entire fatigue life of a welded joint is spent in the crack propagation phase.

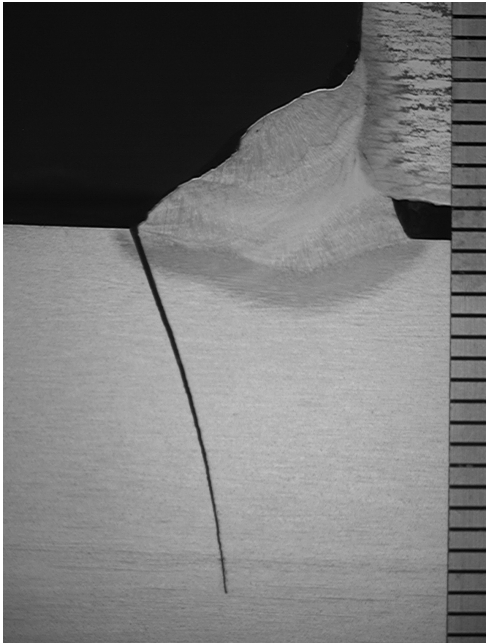


Figure 7.8 Fatigue failure from the toe of a fillet weld (© TWI) (scale bar is in millimetres).

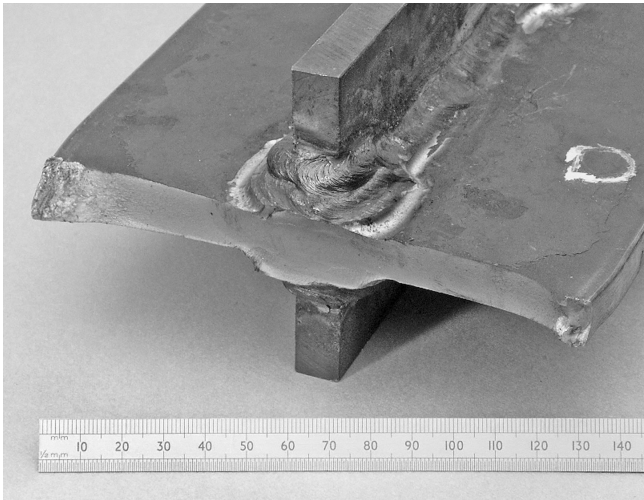


Figure 7.9 Fatigue failure initiating from a welded attachment (© TWI).

7.3.3 Fatigue crack propagation

For steels of different yield strengths under exactly the same conditions, a fatigue crack extends the same (very small amount) in a cycle. Because steels achieve different yield strengths through alloying and heat treatment and, therefore, exhibit different microstructures, fatigue crack propagation in steels is largely independent of steel microstructure. The fatigue crack grows at right angles to the applied stress, right through parent material, weld metal and the heat-affected zone with little regard for microstructure. Fracture mechanics provides a rigorous framework for describing fatigue crack propagation (see Chapter 12, where this is discussed in detail).

When comparing the fatigue behaviour of joints made from low strength and high strength steels, in both cases the joint geometry introduces a similar stress concentration. There is no initiation phase in any welded joint, irrespective of material strength level, and crack propagation occupies the whole fatigue life. As the rate of crack propagation does not depend on material strength, the fatigue behaviour of welded joints of the same geometry is the same for all grades of structural steel.

7.3.4 Residual stress

In Section 3.3 we explained that residual stresses were introduced when welding by the contraction of the weld metal when cooling. As a first approximation, the level of residual stress is considered to be tensile and at the material yield stress, both parallel and at right angles to the weld direction. We have seen that fatigue cracks start to propagate from stress concentrations associated with the welded joint, most commonly, at the weld toe. The fatigue crack starts to grow, therefore, in a region where the stress level is already high and tensile as a result of the welding operation, even before the applied cyclic stresses. This critical region experiences a stress cycle that is markedly different from the applied stress cycle.

Consider an alternating stress cycle (i.e. one that has equal tensile and compressive excursions about a zero mean stress) applied to a welded joint where the material properties at the weld toe may be represented by a simple stress–strain curve, Figure 7.10. Before the fatigue cycle commences, i.e. at zero applied stress, the material at

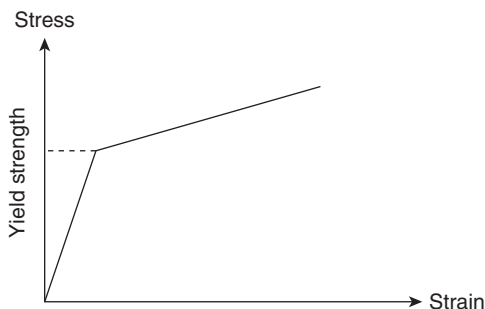


Figure 7.10 Stress–strain curve approximation of the material at the weld toe.

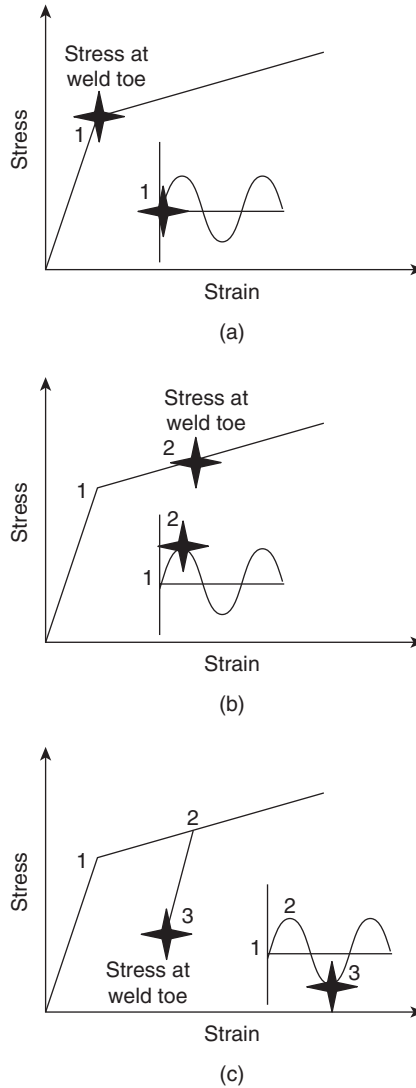


Figure 7.11 Stress at the weld toe (a) before fatigue cycling; (b) at the maximum stress in the fatigue cycle; and (c) at the minimum stress in the fatigue cycle.

the weld toe is already at the tensile yield strength, Figure 7.11(a). As the fatigue cycle commences, further tensile stresses are applied which causes the stress at the weld toe to increase owing to work hardening, until the maximum stress in the cycle is reached. The stress at the weld toe is then as shown in Figure 7.11(b). After reaching the maximum, the applied stress is then reduced causing the stress at the weld toe to reduce elastically, as shown in Figure 7.11(c). At the minimum stress in the fatigue

cycle, the stress at the weld toe has decreased by an amount equal to the stress range in the stress cycle. As the applied cyclic stress is increased again, the stress at the weld also increases until, once again, it reaches the position shown in Figure 7.11(b) at the maximum stress in the fatigue cycle. The critical weld toe region therefore experiences a fatigue cycle that is always in the tensile regime, despite the applied cyclic load being partly compressive.

The illustration in Figure 7.11 may be repeated for other forms of applied fatigue cycle, such as wholly tensile, zero to tension, and other tension-compression cycles (see 6.2). In each instance, it can be shown that the weld toe region experiences a similar stress cycle. As the stress range in fatigue loading is usually much less than the material yield strength, the stresses experienced at the toe in an as-welded joint are almost always entirely in the tensile region, even if the applied stress cycle is partly compressive. Even wholly compressive applied fatigue cycles give rise to tensile stresses at the weld toe and may result in fatigue failure.

The presence of high tensile residual stresses is therefore the reason why the fatigue behaviour of an as-welded joint depends only on the applied stress range and is independent of the mean stress in the applied fatigue cycle.

7.4 Fatigue design of welded joints

In the 1950s and 1960s, extensive experimental fatigue testing programmes on welded joints were carried out. The vast majority of these tests were carried out under axial loading and, therefore, the stress was determined simply by dividing the applied load by the cross sectional area of the plate. In essence, the stress calculated in this way is the nominal stress in the plate, and does not include the stress concentration effect of the joint. Design using nominal applied stress is well established and is used extensively.

7.4.1 *Design using nominal stress and fatigue joint classification*

Extensive sets of experimental fatigue data on welded joints were interpreted by considering the results from each joint geometry in turn and Figure 7.12 shows idealised results from a typical joint. There were often 100 to 200 test results for each joint geometry and the data exhibited a degree of scatter, as would be expected for any body of fatigue data. Simple regression analysis was used to establish a mean line for the data. This, however, should not be used for design as it implies that a joint in service would have a 50% chance of failure before the indicated life had been reached and, therefore, only a 50% chance of exceeding the indicated life. For design purposes, the S–N curve corresponding to the lower limit of the experimental data is needed. There are many ways of estimating the lower limit of the data, but a consensus was reached that a simple statistical approach should be used. The results were assumed to be distributed about the mean line in a way described statistically as a ‘normal’ distribution. This enabled the standard deviation about the mean to be

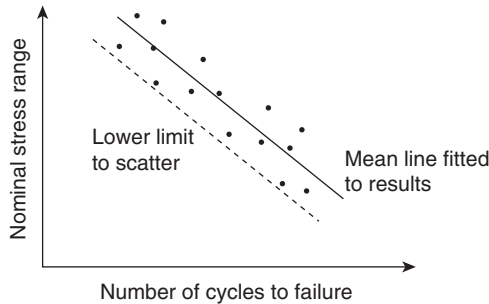


Figure 7.12 Idealised compilation of results for one joint geometry.

calculated from the data, and the design line taken to be the ‘mean minus two standard deviations’ line. In crude terms, this implies that a joint in service has a 97.5% probability of exceeding the design life. Satisfactory engineering experience of using this design approach for some 40 years indicates that the probability of survival is greater than 97.5% and the statistical approach has proved effective, even though the mathematics may not be entirely rigorous.

By using similar statistical analyses of the results from many different joint geometries it was possible to establish design S–N curves for the many individual joint geometries. As there are perhaps 50 or 60 different joint geometries in common use, it would be unwieldy to have such a large number of design S–N curves in a compendium. A simplification was, therefore, made by grouping together those joints geometries with similar design lines and using a single design line for all the joints in the group. These groups are generally known as joint classes or categories, and most national and international codes identify some 10–12 classes. There are, however, detailed differences between the various codes in how individual joint geometries are assigned to a class.

In this way, fatigue design using nominal stress becomes a process of identifying the joint class for each potential failure location, based on joint geometry and direction of applied stress, and then applying the appropriate S–N curve. The nominal applied stress range is selected on the Y-axis of the S–N curve and the corresponding number of cycles, determined from the design S–N curve, read off from the X-axis. As an example, Figure 7.13 shows schematically the design S–N curves for two joint geometries, whereas Table 7.1 summarises some of the most common weld joint designs from three different classification systems. Fatigue joint classifications are given in BS 7608 and in DNV RP C203, and both are commonly used. A similar approach is adopted by the Eurocode documents.

To give some examples, the classification scheme shows that the design S–N curve for a butt weld depends critically on how it is made, whether from one or two sides, using backing or not, and the orientation of the weld relative to the applied loading. The design S–N curve to be used for an attachment welded to the surface of a stressed

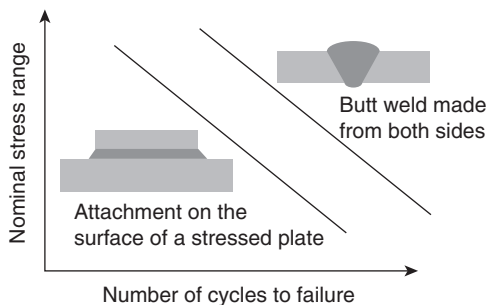


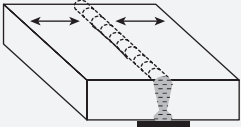
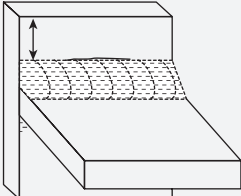
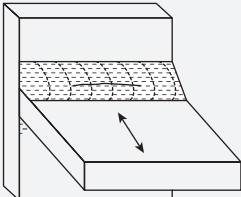
Figure 7.13 Schematic diagram showing the design S–N curves for two joint geometries.

Table 7.1 Some of the most common fatigue weld joint classifications. Other classifications for welded, unwelded and bolted materials are included in the source references

Weld	Description	DNV classification	BS 7608 classification	Eurocode classification
	Automated double-sided butt weld parallel to the applied stress, no start/stops	C1	C	112
	Transverse double-sided butt welds in plate, smooth transitions and no stop/starts.	D	D	90
	Transverse double-sided butt welds in plate made on site.	E	E	80

Continued

Table 7.1 Continued

Weld	Description	DNV classification	BS 7608 classification	Eurocode classification
	Transverse butt weld with temporary or permanent backing (no tack welds)	F	F	71
	Transverse fillet welded attachment or stiffener on a plate under load.	F	F2	80
	Fillet weld under load, expected to crack at the weld root through the throat.	W3	W	36

plane depends on the size and length of the welds on the attachment, and the position of the attachment with respect to the plate edge.

Additionally, the design S–N curve for a joint may be influenced by the presence of a corrosive environment, such as seawater, and whether cathodic protection is applied. The design curve may also be affected by the ratio of axial-to-bending stress, and by the thickness of the joint where this exceeds typically 25 mm for welded plates. Because fatigue performance is not dependent on steel strength, the design S–N curves are used for all steel grades.

7.4.2 Design using hot spot stress

The concept of a ‘nominal’ stress is not appropriate for all joint geometries. For example, jacket structures used for fixed offshore platforms employ a lattice of tubular members that intersect at nodes. Figure 7.14 illustrates a ‘T’ nodal joint where the brace (or stem of the ‘T’) is loaded axially. Local bending is introduced in the chord

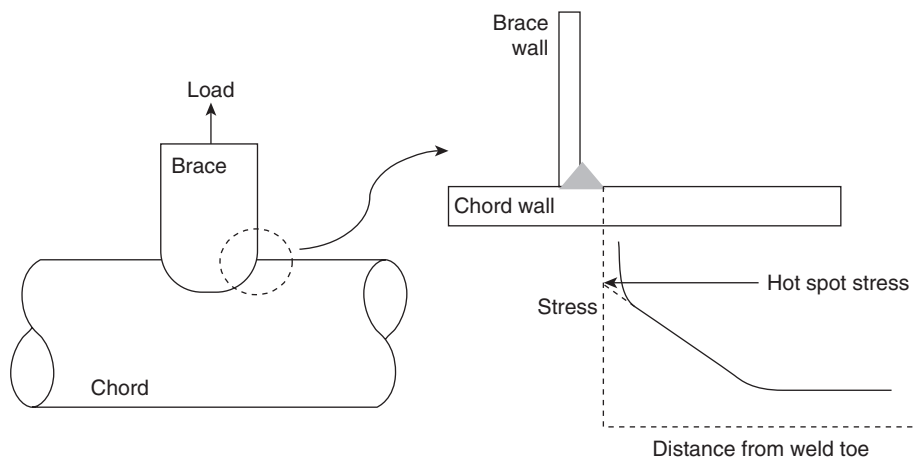


Figure 7.14 Hot spot stress in a tubular connection, defined from the stress distribution near the weld toe.

or main member which gives rise to a stress in the chord that increases very rapidly as the joint is approached. For many tubular joints, the maximum stress in the region of the joint occurs in the main chord member, even though the load is applied through the bracing member, as shown in Figure 7.14. There is, therefore, no clear concept of a 'nominal' stress in the chord.

Measurements made using strain gauges near the intersection of the members showed that the local surface stress in the chord wall increased very rapidly as the weld toe was approached and it was impossible to measure the maximum stress attained. There was, however, a region where the local stress increased linearly before the rapid increase in stress as the weld toe was approached. The maximum stress, or hot spot stress, was therefore somewhat arbitrarily defined as the stress calculated by extrapolating the linear region of stress to the weld toe as shown in Figure 7.14. It should be noted that there is a complex stress distribution introduced in the chord and a hot spot stress can be determined for all positions around the joint. In many cases, the maximum hot spot occurs at the weld saddle, i.e. at the 3 and 9 o'clock positions with respect to the length of the chord.

Fatigue tests on welded tubular connections (Figure 7.15) showed that fatigue cracks initiate at the position of maximum hot-spot stress. Fatigue data for tubular joints are therefore presented on S–N curves, which use the hot-spot stress range as the definition of stress. Furthermore, it was shown that tests on a wide range of tubular joint geometries under different loading conditions, when expressed in terms of hot-spot, could all be represented by a single design curve.

More recently, the concept of hot-spot stress has been used as an alternative to the nominal stress for the design of welded plate structures. This takes advantage of the capability of finite-element analysis to determine the detailed stress distribution in



Figure 7.15 Fatigue test on a tubular welded connection, showing strain gauges attached to record the stresses at different locations on the structure (© TWI).

the vicinity of all weld-joint geometries. Care needs to be taken when interpreting local stresses to define hot-spot stress, but the main advantage is that when hot spot stress range is used, a single S–N design curve can be used for all joint geometries where failure occurs from the weld toe.

7.5 Stress histories of real structures: variable amplitude loading

In our description of fatigue behaviour, we have so far only considered simple sine wave loading, such as that shown in Figure 7.16. This form of cyclic loading is also known as constant amplitude loading. Real welded structures, however, experience loads that vary in time in a much more complicated way. Constant amplitude loading is equivalent to a bridge that only carries vehicles of one weight or an offshore structure always being struck by waves of the same height travelling in the same direction, or haulage trucks always carrying the same load each time. The stress cycle experienced by a bridge actually depends on the size of the vehicle it is carrying; that by an offshore structure on the size and direction of the wave striking it; and that on a haulage truck on the amount of load it carries (Figure 7.17). The stress history experienced by a real structure is therefore very irregular and may look like that shown in Figure 7.18. This form of loading is known as variable amplitude loading.

Welded joints subject to variable amplitude loading may still be designed using the appropriate design S–N curve, and two steps in the process are described next.

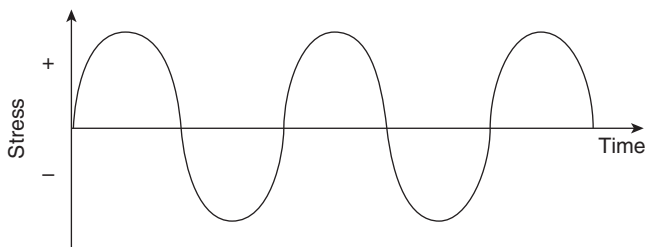


Figure 7.16 Stress cycling from tension (+) to compression (-).



Figure 7.17 Welds in service are subjected to complex loading histories, such as this haulage truck on a construction site (© TWI).

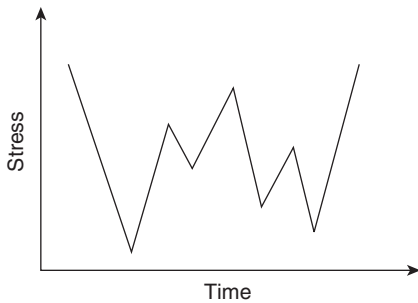


Figure 7.18 Idealised stress history experienced by a real structure.

7.5.1 Equivalent constant amplitude stress history

The variable amplitude stress history is first converted into equivalent blocks of constant amplitude cycling and there are several techniques available to do this. This is called stress cycle counting and, although all stress cycle counting methods seem somewhat artificial, they do indeed work. The most commonly used is the 'reservoir' stress counting method.

The full stress history is arranged such that the stress starts at its maximum value in the entire cycle. The stress history is then considered as the bed of a reservoir which is filled with water, as shown in Figure 7.19a. The minimum stress in the history is then found, point B, and an imaginary plug is removed. The water then drains away leaving new mini reservoirs where water is entrapped, as shown in Figure 7.19b, and we count one stress cycle, AB. The next lowest stress is taken, in this case point D, and another imaginary plug is removed, allowing water to drain away again. We then count a cycle of stress, CD, and continue the process until the entire reservoir is drained. In this way, we break down the variable amplitude stress history into a series of constant amplitude stress cycles, Table 7.2.

An alternative method is the 'rainflow' stress cycle counting technique in which the stress history is turned through 90° to resemble the roof of a pagoda. A raindrop is imagined to fall down the roof and a set of rules enables 'positive-going' and

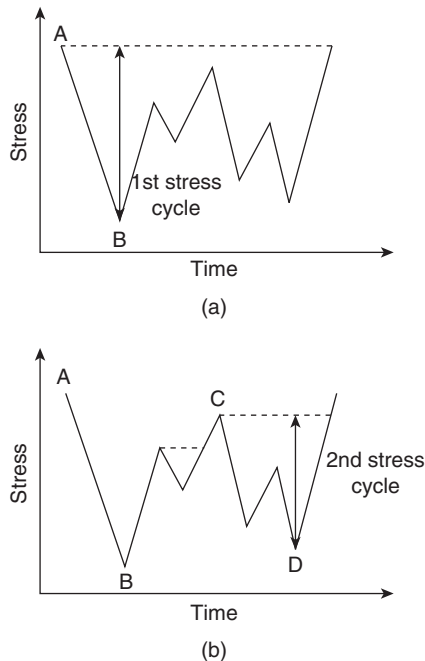


Figure 7.19 The reservoir counting method: (a) the reservoir is filled and (b) drained from the lowest point to identify the cycles to count.

Table 7.2 Fatigue stress spectrum for variable amplitude loading, collated as a set of different constant amplitude stress ranges and the associated number of cycles of each stress range

Stress range	Applied number of cycles
$\Delta\sigma_1$	n_1
$\Delta\sigma_2$	n_2
$\Delta\sigma_3$	n_3
etc.	etc.

‘negative-going’ half cycles to be identified and then matched to define stress cycles. Again, the full stress history is broken down into a series of equivalent constant amplitude stress cycles. Although slightly more complex in principle, rainflow counting has the advantage of evaluating the stress history in ‘real time’ and does not need the entire stress history to be known before counting can be done, as is the case for reservoir counting. Rainflow counting is, therefore, particularly useful for analysis of strain gauge data as it is obtained from a real structure. Both rainflow and reservoir counting methods produce the same breakdown into constant amplitude stress cycles, and ultimately give the same fatigue spectrum.

7.5.2 Adding the damage caused by constant amplitude cycles

The stress cycle counting technique breaks down the variable amplitude loading into a series of constant amplitude stress cycles, as shown in Table 7.2. Each one of these constant amplitude stress cycles is then considered in turn, together with the appropriate design S–N curve, Figure 7.20.

For the first constant amplitude stress range, $\Delta\sigma_1$ the allowable number of cycles is N_1 . The number of cycles actually applied at this stress range, determined by the cycle counting technique is n_1 . We assume therefore that only a proportion of the total life is exhausted at this level of stress range, and that the fraction of life used is equal to n_1/N_1 . This process is repeated for the second stress range, $\Delta\sigma_2$, and establish that the proportion of life used is n_2/N_2 , where N_2 corresponds to the design life at a stress range $\Delta\sigma_2$. We can therefore build up a table of the damage associated with each part of the fatigue stress spectrum as shown in Table 7.3. For an acceptable design, the total of all the fractions of life must be less than one.

For a simple example, consider a structure which experiences 40 000 stress cycles equal to 200 MPa and 20 000 stress cycles equal to 100 MPa. We assume that, for this case, the S–N curve of the weld joints in the structure gives a lifetime N of 50 000 for $S = 200$ MPa and a lifetime N of 200 000 for $S = 100$ MPa. Therefore, the stress cycles experienced at 200 MPa cause damage of $40\,000/50\,000 = 0.8$ (or 80% of the maximum number of cycles at this stress range). The cycles at 100 MPa contribute

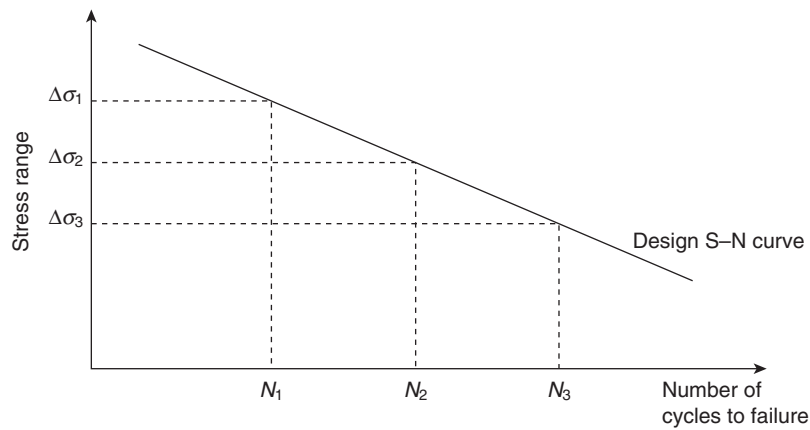


Figure 7.20 Using the design S–N curve to calculate the damage from each stress range in the variable amplitude stress spectrum.

Table 7.3 Fatigue stress spectrum for variable amplitude loading, and the associated damage from each component of stress

Stress range	Applied number of cycles	Allowable cycles	Fraction of life used
$\Delta\sigma_1$	n_1	N_1	n_1/N_1
$\Delta\sigma_2$	n_2	N_2	n_2/N_2
$\Delta\sigma_3$	n_3	N_3	n_3/N_3
etc.	etc.	etc.	etc.

damage of $20\,000/200\,000 = 0.1$ (or 10% of the maximum number at this stress range). The total damage from both of these parts of the stress spectrum is $0.8 + 0.1 = 0.9$ (or $80\% + 10\% = 90\%$). Therefore, the structure has not exceeded its fatigue design limit because the total is less than one.

In this approach, it is assumed that each set of constant amplitude cycling causes a certain amount of fatigue damage, and this damage is simply added together. The technique is therefore known as the linear cumulative damage technique. It is also known as Miner’s rule, named after the engineer who proposed it. In more mathematical terms, Miner’s rule can be expressed as:

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \dots = \sum \frac{n_i}{N_i} \leq 1$$

There is considerable research in progress regarding which value of the summation in Miner’s rule is appropriate. For some cases, where the structure has no

redundancy and the consequences of failure are very serious, a lower value, typically 0.5, may be used instead of 1. Additionally, some codes assume that the design S–N curve changes slope to become less steep at long lives. The use of Miner’s rule assumes that there are no stress cycle interaction effects, i.e. the damage caused by one cycle does not depend in any way on the previous cycle. We know that this is a very simplistic assumption because a single large stress cycle for example may cause the subsequent cycles to be less damaging; this is also known as crack retardation.

7.6 Fatigue of welded aluminium

The fatigue behaviour of welded aluminium joints broadly follows that of welded steel joints in that:

- the fatigue strength of welded aluminium is much lower than that of unwelded material;
- failure occurs by the initiation of a crack at a stress raiser such as a weld toe, and this crack grows steadily and progressively across the load-bearing area;
- fatigue strength of a welded aluminium joint does not depend on the aluminium yield strength; and
- stress range is used to characterise performance.

Detailed design guidance is given in *Eurocode 9 Design of Aluminium Structures*. Welded aluminium joints are placed into categories depending on their geometry and potential failure location and a design S–N curve is provided for each category. This allocation to categories is directly analogous with the classification scheme used for steel joints, but there are small differences in the treatment of individual joint geometries.

The allowable stress ranges for aluminium joints are much lower than those for steel joints; as a rule of thumb the allowable stress range for an aluminium joint is about a third of that allowed for a similar steel joint. Therefore if aluminium is to be used for applications that are to be subjected to cyclic loads, the fatigue design of the joint is of utmost importance.

7.7 Conclusions

The fatigue performance of a welded steel joint is:

- a) much poorer than unwelded material;
- b) does not depend on steel strength level; and
- c) is controlled by stress range and is independent of mean stress, owing to the presence of welding residual stresses.

Realistic complex stress histories are assessed by using a stress cycle counting method to define an equivalent constant amplitude stress history. Miner’s rule of linear cumulative damage is then applied.

7.8 Sources of further information and advice

- BS EN 1993-1-9 (2005) *Eurocode 3. Design of steel structures – fatigue*, British Standards Institution, ISBN 978 0 580 66400 7.
- BS EN 1999-1-3 (2007) *Eurocode 9. Design of aluminium structures – structures susceptible to fatigue*, British Standards Institution, ISBN 978 0 580 70199 3.
- BS 7608 (1993) *Code of practice for fatigue design and assessment of steel structures*, British Standards Institution.
- DNV-RP-C203, (2011) *Fatigue design of offshore steel structures*, Det Norske Veritas.
- Gurney, T. R. (1979) *Fatigue of welded structures*, CUP Archive, ISBN-10: 0521225582, ISBN-13: 978-0521225588.
- Hobbacher, A. (1996) *Fatigue design of welded joints and components: recommendations of IIW Joint Working Group XIII–XV*, Woodhead Publishing, ISBN-10: 1855733153, ISBN-13: 978-1855733152
- Maddox, S. J. (1991) *Fatigue strength of welded structures, 2nd edition*, Woodhead Publishing, ISBN-10: 1855730138, ISBN-13: 978-1855730137.
- Niemi, E., Fricke, W. and Maddox, S.J. (2006) *Fatigue analysis of welded components: designer's guide to the hot-spot stress approach*, Woodhead Publishing Ltd, ISBN-10: 1845691245, ISBN-13: 978-1845691240.
- Niemi, E., Fricke, W. and Maddox, S. J. (2006) *Structural hot spot stress approach to fatigue analysis of welded components*, IIW Best Practice Document XV-1090r1-03.
- Radaj, D. (1990) *Design and analysis of fatigue resistant welded structures*, Woodhead Publishing, ISBN-10: 1855730049, ISBN-13: 978-1855730045.

Failure modes and analysis in metals

8

8.1 Introduction

In the previous chapters of this book we discuss the importance of understanding the strength of materials from the stress–strain curve, we described the use of fracture mechanics to explain brittle fracture, and we considered fatigue failure in welded and unwelded materials. The three major failure modes in welds: ductile failure, brittle fracture and fatigue failure have been described. However, service failures often involve one or more of these failure modes and examination of the fracture surface enables each specific failure mechanism to be identified. Although the modes of failure that can be recognised from a fracture surface only tell part of the story of the failure, they can help point towards a possible cause of failure.

8.2 Ductile failure

Ductile failure is also known as plastic collapse, general yielding or ductile overload, and is the failure mode that occurs when a material is simply loaded to beyond its ultimate tensile strength (see Chapter 4). Purely ductile failures tend to be rare in service, as the first principle of structural design is to ensure that a component does not exceed its yield strength. This is usually guaranteed by applying a factor of safety on the maximum allowable loading, meaning that the highest stresses should be well below yield at all times. If a purely ductile failure does occur, it is usually because of an error in design, or because, in practice, the actual loading is higher than anticipated (such as excessively high wind loading during a hurricane or load redistribution after failure of another member in the structure).

Alternatively, the material's yield strength may be lower than expected. It is possible to find ductile failures occurring after a fire where the excessive heat has reduced a structure's yield strength to an extent that it has become overloaded by stresses that are well below yield when the structure is at room temperature. In many instances, ductile failure is the secondary cause of failure, after the load-bearing area of a structure has been reduced by some other means. A reduction in the load-bearing cross section arises once a fatigue crack has propagated to a significant size, or when wall thinning has occurred as a result of erosion or corrosion.

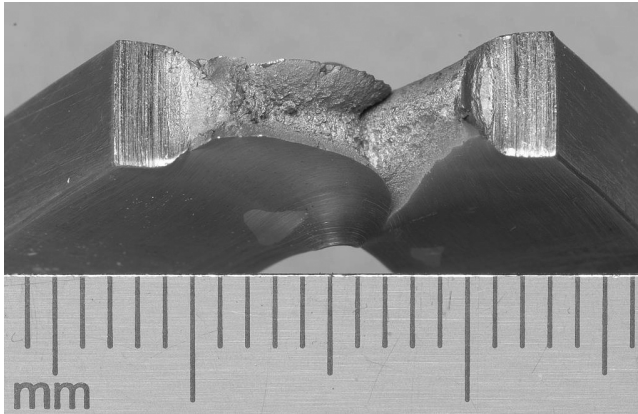


Figure 8.1 Charpy test specimen after test having failed in a ductile manner, showing significant amounts of tearing and deformation on the fracture surface, along with shear lips at the outer surfaces (© TWI).

A ductile failure shows the appearance of yielding and plastic deformation leading up to the final failure. The fracture appearance is rough and torn (Figure 8.1). After a ductile failure, it is not possible to ‘fit’ the broken pieces back together again (as may be possible for a brittle fracture). In a pressure vessel or pipeline under pressure, some bulging may precede the final ductile failure (Figure 8.2).

In a ductile failure, there are characteristic ductile shear surfaces angled at 45° to the applied load (Figure 8.3). These 45° planes correspond to the planes of maximum shear stress in a member under load. They are also sometimes known as ‘shear lips’ (Figure 8.1). Classic ductile failure is exhibited in a tensile test specimen after test, where the parallel sided gauge has necked down at the failure point and the final fracture shows a ‘cup-and-cone’ shape (Figure 8.4). The ‘cup’ part of this type of fracture is formed of a circle of material at 45° to the specimen axis and is related to the 45° shear planes characteristic of ductile failure.

8.3 Brittle fracture

Brittle fracture is a failure mode of great concern to welding engineers. Once initiated, it is a fast and unstable type of fracture. There is no advance warning and the consequences can be catastrophic. Brittle cracks propagate at the speed of sound in steel and are accompanied by a loud bang. The causes of brittle fracture and fracture mechanics are discussed in Chapter 5.

A brittle fracture occurs at right angles to the applied loading and shows little plastic deformation (Figure 8.5). The two halves of a brittle fracture might ‘fit’ back together again because the material has deformed so little. The fracture surface may

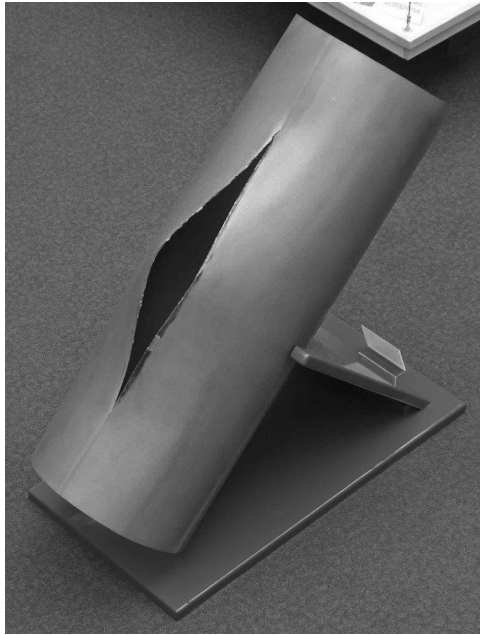


Figure 8.2 Section of pipe which has undergone a pressure test. The component has failed in a ductile manner, as indicated by the bulging around the seam weld, and roughness of the torn weld fracture surface (© TWI).

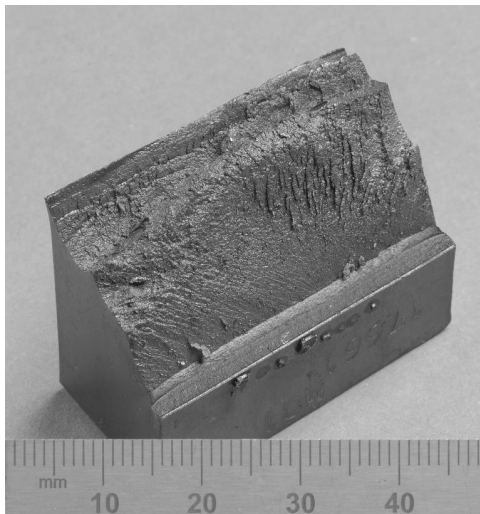


Figure 8.3 Notched tension test specimen in steel after failure, showing final ductile failure surfaces at roughly 45° to the direction of loading during the test (© TWI).

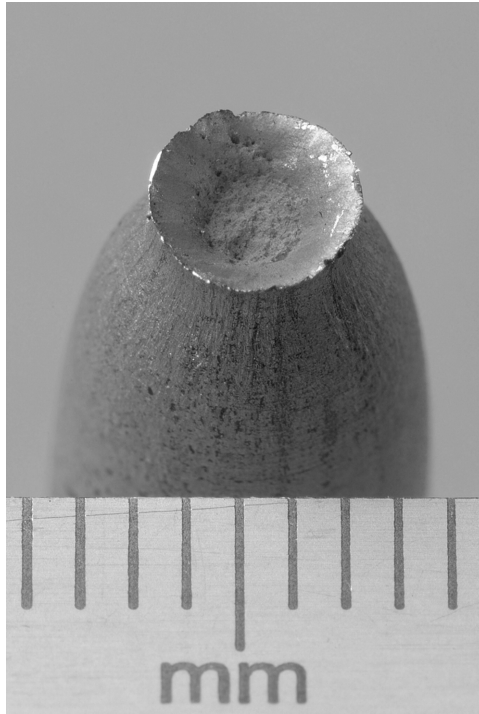


Figure 8.4 Tensile test specimen after test, showing necking down of the gauge close to the failure point, and the 'cup' side of a cup-and-cone type fracture surface (© TWI).

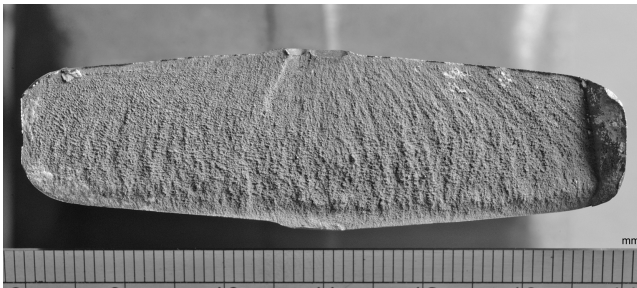


Figure 8.5 Brittle fracture showing slight river lines pointing to a hard stamp mark at the middle of the upper surface. The fracture has occurred at right angles to the applied loading through this bar (© TWI).



Figure 8.6 A piece salvaged from the John Thompson pressure vessel failure in 1965, now on display at TWI, Cambridge. The thick section forging shows a classic brittle fracture appearance with river lines pointing towards an initiation point at or beyond the location that the lifting lug has been attached (© TWI).

show features called river lines (or chevron marks), which point back to the initiation point of the fracture (Figure 8.6). These river lines can help to show which feature or defect on the fracture face initiated the brittle fracture. Because the river lines point to the initiation point, the direction of the crack propagation is the opposite direction. This is useful, for instance when a pressurised component fails in a brittle manner and fractures into many pieces for identifying the direction in which the crack propagated and, thus, piecing together the sequence of the failure by tracing the direction of the travelling cracks around each piece back to the point where they initiated.

Brittle fracture may propagate in one of two ways through the steel's grain structure. It is common for brittle fracture to propagate directly through the microstructure, effectively ignoring the grain boundaries in the steel; this is called 'transgranular' fracture. When a transgranular brittle fracture surface is observed under high magnification, there are microscopic river lines that point towards the fracture initiation point, in an equivalent manner to the macroscopic river lines visibly

seen on a brittle fracture. Therefore, observing the region of brittle fracture initiation under a scanning electron microscope (SEM) can help to identify the initiating defect at a microscopic level (see 8.5). If the steel contains impurities such as sulphur or phosphorus, or has become embrittled such that the grain boundaries are weakened, the alternative fracture propagation path follows the grain boundaries through the steel structure; this is called 'intergranular' fracture. The low toughness grain boundaries offer the least resistance to the propagating crack. Using SEM, the blocky appearance of individual grains can be resolved on the fracture surface when intergranular fracture has occurred.

Intergranular fracture often occurred in older style steels because of their larger grain sizes and higher sulphur levels which led to grain boundaries with low ductility. During Charpy testing of these steels, a brittle fracture could sometimes appear shiny or faceted like sparkly crystals, owing to the intergranular nature of the impact fracture. Before brittle failure was fully understood, brittle fracture as evident from a Charpy specimen was thought to be caused by the steel having 'crystallised'. However, this is meaningless as all structural metals are crystalline, and remain so throughout their lives. The intergranular fracture appearance in this instance was a consequence of the high loading rate and the large grains with weakened grain boundaries. Most modern steels contain low amounts of sulphur and have a fine grain size with transgranular fractures that make them dull in appearance.

8.4 Fatigue failure

The full details of fatigue are covered in Chapters 6 and 7, particularly its significance for welds and its avoidance by design. The intention of this chapter is to provide guidance on how to recognise the appearance of fatigue cracking on a fracture surface. Fatigue cracks grow slowly over time under the repeated action of a cyclic load, below the material's yield strength. There may therefore be time to detect the growing crack during the service of the component. A fatigue crack propagates steadily until it has reached a size sufficient to cause failure, with final failure occurring by a secondary failure mode. Either the fatigue crack reduces the load-bearing cross section to such an extent that ductile overload occurs (Figure 8.7), or the crack becomes large enough to initiate a brittle fracture. The final failure mode depends on the fracture toughness and strength of the material.

Fatigue cracks are oriented at 90° to the applied load and can be recognised by their flat and smooth appearance (Figure 8.8). The extent of the crack at any given time is bounded by a curve, centred on the initiation point for the fatigue crack. In a fatigue test specimen where the applied stress range is constant, the fatigue crack surface can seem fairly featureless. For a fatigue failure in service where there have been changes in loading condition, such as periods of shutdown, or occasional peak loads between the normal range of cyclic loading, then beachmarks may be seen on the fracture surface (Figure 8.9). Beachmarks are so named because they may resemble the line of seaweed left by the high tide on the beach. When a fatigue crack propagates and then experiences a change in its loading conditions, perhaps a few

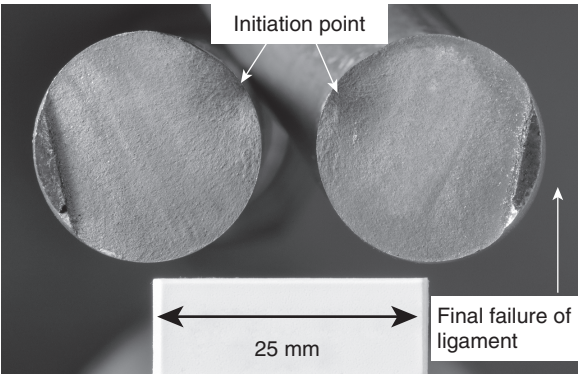


Figure 8.7 Fatigue cracking across a steel bar. The two halves of the same failure are shown. The final ligament has then failed by overload (© TWI).

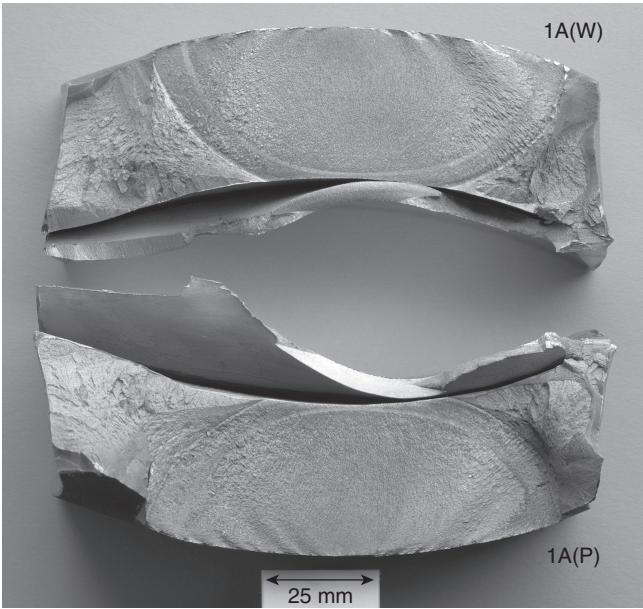


Figure 8.8 Fatigue failure of a test specimen from a carbon steel with a corrosion-resistant alloy liner. The fracture surface shows a flat and smooth fatigue crack bound by a curve in the central region, and final ductile failure at the sides of the failure. The cladding has debonded and failed in a ductile manner (© TWI).

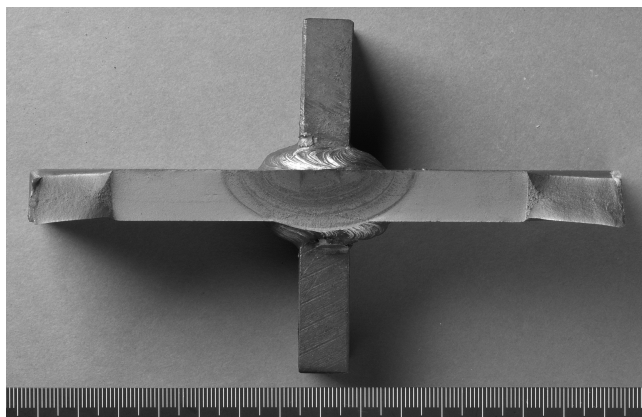


Figure 8.9 Beachmarks on a fatigue fracture surface in steel. The beachmarks here are also associated with some corrosion, suggesting that they have been exposed to the environment for some time (© TWI) (scale bar is in millimetres).

excessively high loads or a shut-down for some time, the extension of the crack resulting from that cycle is different to the normal crack extension per cycle. As the crack then continues to propagate this becomes noticeable on the crack surface, as a beachmark indicating the shape of the fatigue crack at that point of excessive load or shutdown condition. Beachmarks therefore represent the position of the crack front at the time a change in conditions occurred. When interpreting a fatigue failure surface, it may be possible to relate individual beachmarks to known events in the structure's load history, in an attempt to determine how long the fatigue crack has been present in the structure.

In some alloys, such as aluminium alloys, the microscopic appearance of a fatigue crack shows individual striations associated with every single stress cycle that has added to the growth of the fatigue crack. These fatigue striations are unfortunately almost never observable in steels, but they can be an invaluable tool for investigating fatigue failures in aluminium alloys, particularly in aircraft accidents. The striations on the fracture surface can be counted, like the rings on a tree stump, to estimate the rate of crack growth per cycle. Whereas beachmarks can be related to significant load changes, individual striations are related to individual stress cycles.

Fatigue cracks on a fracture surface also frequently show 'ratchet marks'. These marks occur when many fatigue cracks initiate from multiple sites close together, and subsequently grow into one single crack. As these initial cracks join up, they are slightly out-of-plane, and the crack breaching these steps between planes causes the ratchet marks. Ratchet marks appear as short lines perpendicular to the crack propagation direction. These can be seen on the top surface of the fracture in Figure 8.8 and at the centre of the initiation point in Figure 8.9.

In the popular press, this type of failure is sometimes referred to as 'metal fatigue', and described in terms as if the material had degraded or 'worn out' over time.

However, fatigue has nothing to do with any change in mechanical properties, but is actually all about crack growth over time.

8.5 Scanning electron microscopy (SEM) of fracture surfaces

Some fracture surfaces can appear rather featureless to the eye, making it difficult to identify the mode of failure from macroscopic features. In these cases, it can be beneficial to examine the fracture surface using SEM to identify the microscopic features of the failure. Using SEM the fracture surface is magnified by up to several orders of magnitude and has a large depth of field, thus allowing a three-dimensional image of fracture surface to be obtained much more effectively than by an optical microscope.

The SEM image of a ductile failure shows a network of cup-and-cone style ridges and dips. This characteristic appearance of ductile failure is called microvoid coalescence (Figure 8.10). Microvoid coalescence occurs when voids form around tiny inclusions in the steel as it stretches open during yielding. These eventually combine together to create larger voids and, eventually, fracture. The ridges between the voids are the last locations of deformation. Sometimes a crack in a ductile material propagates under stable ductile tearing for a few millimetres before final rupture or even a brittle fracture. When this stable ductile crack extension is viewed under the microscope, microvoid coalescence is identified as the failure mechanism.

Brittle fracture appears faceted under the microscope, with the fracture facets generally cutting through the grains in a transgranular manner. On these microscopic

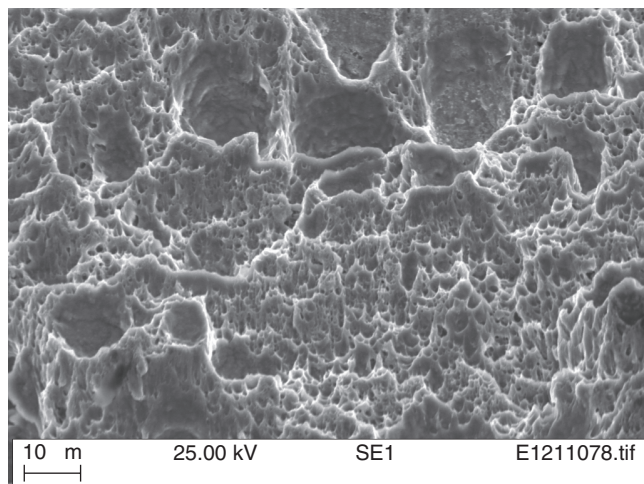


Figure 8.10 Ductile fracture surface as imaged using a scanning electron microscope showing microvoid coalescence in between larger ridges of deformed material (© TWI).

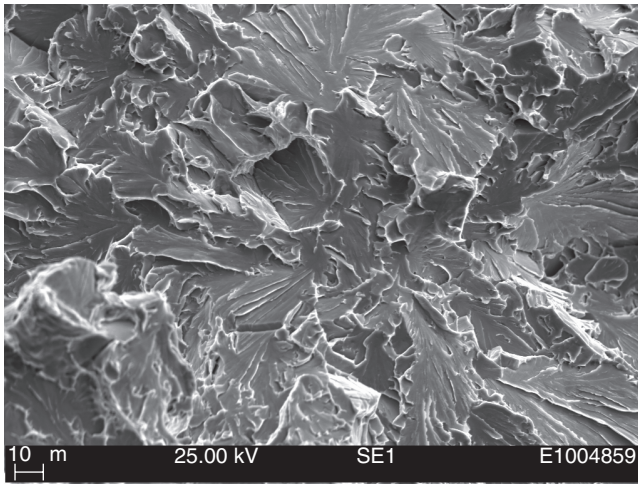


Figure 8.11 Brittle fracture under the scanning electron microscope showing river lines on transgranular facets of the brittle fracture, pointing to the initiation point in the centre of the image (© TWI).

fracture planes it is possible to observe river lines which, like the macroscopic river lines on a brittle fracture, point towards the initiation point (Figure 8.11). The initiating defect or feature might be identified under the microscope if it is not clear from the initial visual inspection of the fracture.

In brittle fractures where intergranular fracture is the crack propagation mechanism, this appears at high magnification to show the flat facets of each grain crystal at angles to one another. The shapes and sizes of the grains along the fracture path are easy to observe.

Fatigue cracks remain fairly flat in appearance even when examined by SEM, but they have a characteristic appearance which has been likened to ‘cottage cheese’, being slightly rippled (as seen at the bottom of Figure 8.12). It is also possible, under greater magnification, to see features like beachmarks (Figure 8.12), and to seek the initiation feature for the fatigue crack, from the central point of the curved fatigue crack. In aluminium alloys, and other metals which can exhibit striations during fatigue crack propagation, these striations are often only resolvable by SEM, and are difficult to see under an optical microscope.

8.6 Interpreting fracture faces

Various approaches are used in interpreting fracture faces, including the use of fracture replicas and Charpy tests.

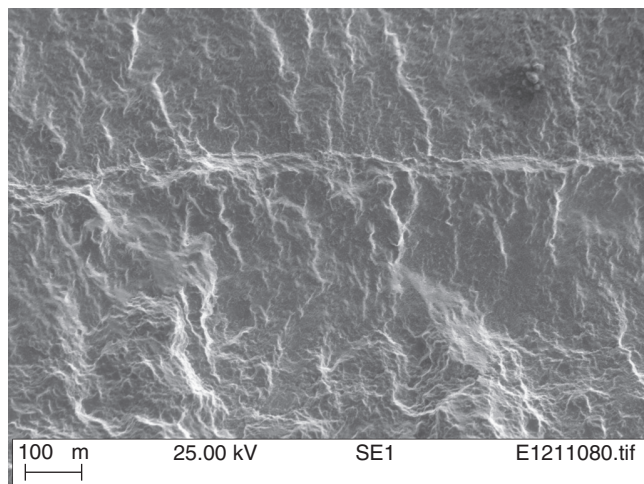


Figure 8.12 SEM image of a fatigue crack, showing a fairly flat surface with a beachmark running horizontally across the middle of the image. Magnification given by the scale marker (© TWI).

8.6.1 Fracture replicas

Although it is always best to examine the fracture surface directly, sometimes it is not possible or practical to do so, e.g., if the fracture surface cannot be removed from its location, or if a permanent record needs to be made of a fracture surface before it is cut for subsequent testing and analysis. In these instances, it is possible to make a plastic replica of the fracture surface.

Replicas of fracture surfaces can be used to learn about the appearance of brittle, fatigue and ductile failures (Figure 8.13). Replicas are cast from the metal components but are made from a durable plastic, which makes them easier to store, transport and handle. Looking at replicas of known failures can help build up experience in interpreting fracture faces, even long after the original fracture was formed. The quality of the replica that is cast depends on the requirements. It is possible to preserve even the microstructural features of the fracture surface if replicated using the highest quality plastic resin and great skill. However, should examination of the replica by SEM be necessary then an electrically conductive plastic is preferred, or the plastic is coated in a conductive material. If the requirement of the replica is simply to preserve the dimensions and macroscopic appearance of the failure then a lower quality resin is sufficient.

8.6.2 Charpy fracture faces

Characterising the appearance of the fracture surface of Charpy test samples, is necessary in order to fully record the results of the tests. When Charpy testing is used



Figure 8.13 TWI's plastic fracture replica set, used during training courses to recognise ductile, brittle and fatigue failure modes (© TWI).

to generate a ductile-to-brittle transition curve, the fracture appearance is described in terms of the proportion of brittle or ductile failure modes present on the fracture face. Some Charpy test standards require the percentage crystallinity (brittle fracture appearance), to be quantified in the test results, whereas others give the percentage shear area (ductile fracture appearance). The amount of the given fracture appearance is usually estimated to the nearest 5%, rather than attempting to quantify the proportions with higher accuracy, because this is sufficient to allow its position on the transition curve to be identified. The appearance of Charpy fractures across a ductile-to-brittle transition curve is shown in Figure 8.14 from a set of specimens tested over a range of temperatures. The test temperature increases from left to right in Figure 8.14, and the percentage crystallinity ranges from 90% on the far left, through 80%, 60%, 20% down to 0% on the far right.

8.6.3 Characteristic features of failure modes

When examining fracture surfaces from components that have failed in service, it can be useful to consider the clues and features that are characteristic of particular failure modes, in order to help interpret how the component failed. Not every fracture face

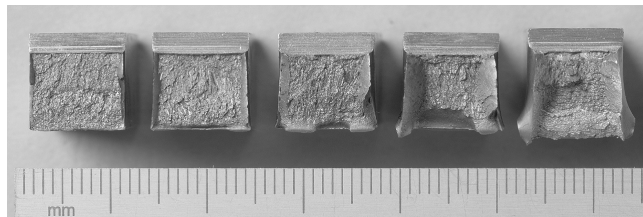


Figure 8.14 The fracture appearance of a selection of Charpy test specimens used to generate a transition curve, tested at successively higher temperatures from left to right. The percentage crystallinity ranges from 90% on the far left, through 80%, 60%, 20% down to 0% on the far right (© TWI).

shows all the typical features for each failure mode, but there are certain features which are characteristic of just one failure mode. If there are fracture planes at 45° to the applied load then ductile failure has played a part, because these only occur when plastic shear planes are formed. Beachmarks are certain evidence of fatigue when they can be seen on flat and smooth fatigue cracks. River lines on a rough fracture surface mean brittle fracture has played a part in the failure. The absence of these features does not mean that these modes have not occurred but when they are present on a fracture face identification of the failure mode(s) is more straightforward.

8.7 Corrosion

Corrosion is often a contributing factor to weld failures, particularly after a significant time in service. General corrosion is usually accounted for in the structure's design in terms of a corrosion allowance: an increase of a few millimetres in the required wall thickness beyond what is needed for load bearing or pressure containment. This type of corrosion in steels is what might commonly be called 'rust' and steel structures weather in the rain and air in a slow and steady fashion. Provided the corrosion allowance has not been exceeded (verified by routine ultrasonic inspection), there should be little risk of ductile failure.

If corrosion occurs unevenly, the steel is corroded much more quickly at small, preferential sites, which may be around welds, owing to their different chemical composition or because they provide crevices. Corrosion under insulation (CUI) is a similar corrosion phenomenon, when water (usually from rain) seeps underneath the thermal insulation around a pressure vessel, causing the wall to corrode relatively rapidly from the outside, hidden from view by the insulation around the vessel. Sometimes it is the corrosive products inside a pressure vessel that cause corrosion in the form of pitting on the inside surface of the steel. These pits can be relatively

deep and act much more like sharp defects than the smooth profile of general wall loss corrosion.

Corrosion can become very dangerous for structural integrity when it accompanies cracking, such as stress corrosion cracking at welds. Particular combinations of alloys and environments are susceptible, for example austenitic stainless steels in chloride (salt water) environments, or carbon steels in sour (hydrogen sulphide) service. Stress corrosion cracking can be very damaging for welds because the main driver for crack propagation is weld residual stress, which is always present in welds, and propagation of stress corrosion cracks can be fairly rapid in service.

Corrosion is often a primary failure mechanism before the secondary failure mode that causes the final failure. There are many corrosion-related and stress-corrosion mechanisms which require careful assessment by an integrity engineer, but are not necessarily the concern of the welding engineer and, therefore, it is not the intention of this book to discuss them in any detail. When corrosion causes excessive wall loss, the structure is at risk of ductile failure, whereas stress corrosion cracking might be an initiating defect for brittle fracture. Corrosion on a fracture surface can sometimes provide hints as to the cause or timescale of failure (such as in Figure 8.9).

8.8 Engineering failure investigations

Being able to recognise the different failure modes from their appearance on a fracture surface can enable the 'story' of the failure to be understood. This is a vital part of any engineering failure investigation. For example, where large fatigue cracks can be seen it can be reasoned that these have been present for some time, and may be the initiating part of the failure. Brittle fractures occur suddenly, but they may be localised if a brittle crack initiates and subsequently arrests or stops once it extends outside the high stress region or into an area of higher toughness or lower stressed material. Ductile features are usually the final parts of the failure. River lines or beachmarks on a brittle or fatigue failure, respectively, indicate the origin of the failure whether that might be a pore, crack or other stress-concentrating feature.

Failed components from service may be heavily corroded by the time they are salvaged, for example, from the sea bed. The corrosion can damage the microscopic appearance of the fracture surface, but the macroscopic features may still be distinguishable enough to identify the modes of failure (Figure 8.15). Sometimes replicas are made of fracture surfaces during a failure investigation so that the macroscopic features of the whole fracture surface is preserved and re-examined after microscopic samples have been cut from it for more detailed testing and analysis.

Performing a full engineering failure investigation is a large and complex task. It is important to understand that recognising the modes of failure (such as fatigue or brittle fracture) is different to identifying the 'cause' of the failure. The evidence from the fracture surface, as well as operating records, fabrication records, eye witness accounts, and mechanical test data all contribute to understanding how the failure occurred and to suggest its root cause.



Figure 8.15 A corroded (rusty) failed bar showing evidence of river lines pointing to an initiation point at the bottom of the image, suggesting a brittle fracture has occurred (© TWI).

8.9 Conclusions

Interpreting the metallurgical failure modes from a fracture surface is only part of a whole failure investigation, but it is often the first analytical step towards identifying the cause or causes of failure. Fatigue cracking leads an investigation towards finding sources of cyclic loading (which could be either mechanical or thermal).

Ductile failure requires identification of the sources of excessive loading as the cause of the failure. Brittle fracture implies that defects were probably present in the component or that the material was embrittled or had low fracture toughness for some reason. The investigation, therefore, follows a different line of enquiry in each of these cases. The evidence from the fracture surface must be consistent with the theories postulated during the failure investigation. Until the mode and cause(s) of failure are identified and understood then it is not possible to put measures in place to prevent similar failures from happening again. Characteristic features of brittle, fatigue and ductile failure can help identify which modes have contributed to the failure. Engineering failures can be catastrophic in terms of loss of life and financial consequences so that recognising particular failure modes is vital in order to understand the cause(s) of failure, and, thus, take steps to avoid other failures. Ensuring the avoidance of failure through fitness-for-service assessment is discussed further in Chapters 11 and 12.

8.10 Sources of further information and advice

- Brooks, C. R. and Choudhury, A. (2001) *Failure analysis of engineering materials*, McGraw–Hill Professional Engineering, ISBN-10: 0071357580, ISBN-13: 978-0071357586.
- Matthews, C. (1998) *A practical guide to engineering failure investigation*, Wiley–Blackwell, ISBN-10: 1860580866, ISBN-13: 978-1860580864.
- McEvily, A. J. (2001) *Metal failures: mechanisms, analysis, prevention*, John Wiley & Sons, ISBN-10: 0471414360, ISBN-13: 978-0471414360.

9.1 Introduction

When considering the ability of a welded joint to withstand loading, whether at high stress, low temperature or under variable stress cycles, it is useful to carry out mechanical testing of small scale weld test samples to assess the performance of the joint. It is common to carry out a range of mechanical tests during weld procedure qualification to demonstrate its suitability, but often it is desirable to perform more specialised testing, particularly for joints that are to be subjected to the most severe service loading.

Some mechanical tests are purely qualitative, such as bend tests or examination of macrosections; they provide a pass or fail result. It is not possible to obtain design data from qualitative tests, but they do tend to be quick, cheap, simple and used often in weld procedure qualification. The results of quantitative mechanical tests on the other hand, can be used to quantify materials properties for use in design or other calculations. Examples are a round bar tensile test and a fracture toughness test.

Some tests might be considered quantitative because a numerical value is generated from the test, but the use of that data is solely for quality control and, therefore, these tests are also considered to be qualitative. Examples are cross-weld tensile tests, hardness tests and Charpy tests.

9.2 Weld procedure qualification

It is important to make sure that the welding procedure specification (WPS) being used to perform a weld gives a sound weld with good mechanical properties. Weld procedures, and some of the problems associated with welding are discussed in Chapter 3. By qualifying the welding procedure, it is possible to demonstrate that welds made using the procedure possess satisfactory properties and are acceptable. The qualification process often involves carrying out a set of mechanical tests on a weld that has been made using the preliminary welding procedure. The weld is also subject to visual examination and non-destructive testing (see Chapter 10). If all the mechanical tests give satisfactory results and the weld is not found to contain significant defects, then the preliminary welding procedure becomes the qualified WPS.

The testing required for a weld procedure qualification depends on which standard the qualification is being carried out to. The most common choice in Europe is

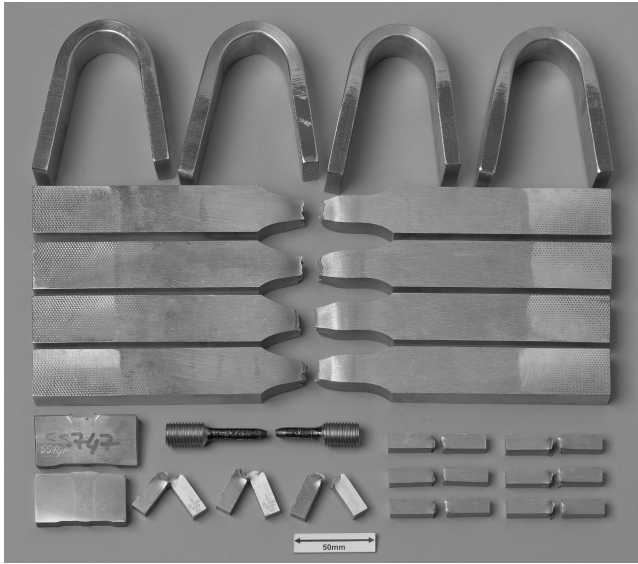


Figure 9.1 Typical mechanical testing for a weld procedure qualification. From top to bottom and left to right; bend tests, cross weld tensile tests, macros, round tensile test, Charpy tests (© TWI).

BS EN ISO 15614 which comes in 13 parts for different materials and welding processes (Part 1 is for arc welds in steels and nickel alloys, Part 2 is for arc welds in aluminium alloys). In the USA the ASME boiler and pressure vessel codes are followed more commonly, with Part IX referring to the welder and weld procedure (and brazing) qualification. Both standards identify the essential variables in the welding procedure (such as welding current, material type, thickness or consumable) and each of these has an associated range of approval, so that one weld procedure qualification record (WPQR) can be used to generate a number of similar WPSs within that range. Changing an essential variable outside its range of approval requires the welding procedure to be requalified. ASME IX requires only tensile and bend tests to be carried out to qualify a butt weld. BS EN ISO 15614 Part 1 requires a far more extensive test programme of visual inspection, radiography or ultrasonic examination, surface crack detection, tensile and bend tests, and macroexamination (Figure 9.1). In certain circumstances Charpy-V impact tests and hardness surveys are also required.

9.3 Bend testing

The bend test is a simple qualitative test on a butt weld to demonstrate soundness and ductility. A strip sample is sliced from the weld and bent, either by pressing into

a curved-shaped former, or bending around a central former using equipment similar to a plumber's pipe bending rig. The bend angle is specified within the test method and can be from 90° to 180° . If the weld joint is less than 12 mm thick then a wider slice is taken and bent with either the weld root or weld face on the outside of the former. In these instances, the tests are known as 'root bend' or 'face bend' tests to distinguish them from 'side bend' tests. Sometimes one or more of each type of sample is tested. The purpose of the test is to deform the weld joint sufficiently to demonstrate its ductility without fracturing. The amount of deformation induced during the test means that any surface-breaking defects present in the root or face bend test specimen are opened up and revealed. Side bend tests reveal buried lack of fusion defects. The test is either pass or fail, with the presence of a defect of more than 3 mm, for instance, being a reason for failing the test.

The precise details of the bend test, such as the former diameter and the bend angle, are given in the appropriate standard or specification. This might be a fabrication standard rather than a specific bend test standard. For example, although bend tests are standardised in ASTM E190, the tests are also described in API 1104 for qualification of pipeline welds, or BS EN ISO 15614 for weld procedure qualification.

Bend tests are simple enough to be performed on the shop floor during production for quality control, rather than needing to be sent away to a test house. This makes bend tests quick to perform, and simple to interpret. Examples of bend test specimens after being successfully tested are shown in Figure 9.1. However, the test is only intended to be an indication of the ductility and soundness of the weld. To quantitatively determine the strength or ductility of a weld, tensile tests are necessary.

9.4 Tensile testing

The tensile test measures the strength of a material (strength is discussed in more detail in Chapters 2 and 4). There are two kinds of tensile test that can be used for welds; the round bar tensile test, and the cross-weld tensile test. The round bar tensile test specimen is machined with a parallel sided round cross-section central region called the gauge, which blends out at the ends to wider grip regions which are often threaded for easier insertion into the testing machine. The gauge is the part of the specimen which is being tested and is homogeneous material (either all parent metal or all weld metal).

A material's strength is independent of material thickness so different size tensile specimens from the same material give the same results in terms of strength. This allows the results of standardised tensile tests to be comparable and reproducible. However, for materials with large grain sizes, such as high heat input weld metals, or non-ferrous alloys, a larger tensile specimen is preferable in order to generate more consistent results that are unaffected by the microstructure.

The test specimen's size and dimensions are usually given by the testing standard, and the gauge diameter and gauge length scale relative to the thickness of the material being tested. It is usual to machine as large a tensile specimen as possible from the

material, up to a gauge diameter of 10 or 12 mm. The gauge length is four or five times the gauge diameter depending on the specifics of the testing standard. The tensile specimen's gauge diameter is measured carefully before testing so that the cross sectional area is known. The length of the specimen's gauge between two marker points is also carefully measured before the test so that the original gauge length is known.

9.4.1 Test procedure

An extensometer is applied across the gauge which measures the extension in the gauge of the specimen as it is loaded under increasing tension (Figure 9.2). Tensile tests can be carried out at any temperature; high temperature gives a lower yield strength, whereas low temperature gives a higher yield strength relative to the room-temperature tensile properties. If the test is performed at a temperature other than ambient temperature it needs to be brought to a stable temperature before the test, and the temperature monitored throughout the test. The tensile specimen is loaded



Figure 9.2 Round tensile test specimen fitted with an extensometer across the gauge during a tensile test (© TWI).

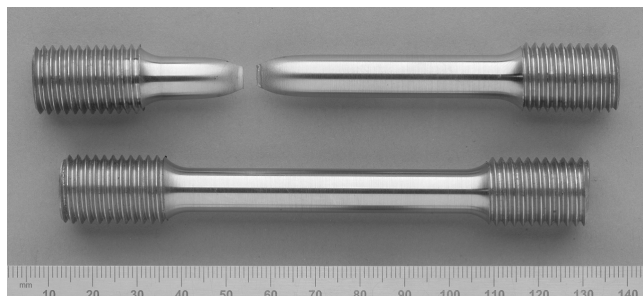


Figure 9.3 Round tensile test specimen as machined before (below) and after tensile testing showing necking around the failure location, and measurable extension relative to the untested gauge length (© TWI).

under uniaxial quasistatic loading, i.e. very slowly increasing load along the same orientation as the test specimen. This type of loading is also sometimes called monotonic loading. The tensile specimen initially extends elastically until the yield point is reached, at which stage permanent deformation of the specimen starts to occur. The loading continues to increase until the specimen starts to neck down at a point within the gauge and, finally, rupture occurs. If the test specimen is loaded at a high strain rate, the material does not have sufficient time to deform in a fully ductile manner, and the test measures a higher strength and a less ductile result.

During the tensile test, the results are continuously logged in terms of the applied load (in N), and the extension of the extensometer (in mm). The load is converted to an applied stress (in MPa or $N\ mm^{-2}$) by dividing by the cross section area of the gauge. The extensometer measurements are converted into strain readings by dividing the increase in length by the original gauge length, which is usually expressed as a percentage. In this way, a stress versus strain curve is generated from the tensile test data. From the stress–strain curve, the yield strength, ultimate tensile strength and the Young's modulus can be determined. These parameters are discussed further in Chapter 4.

After the test, the diameter of the narrowest point of the necked region is measured and the broken test pieces are placed next to each other again to measure the final deformed length of the gauge between the marker points at the point of failure (Figure 9.3). The percentage reduction in area is the difference between the gauge diameter after test and before test, expressed as a percentage of the original gauge diameter. Similarly, the percentage elongation is the increase in length of the gauge expressed as a percentage of the original gauge length. The reduction in area and the percentage elongation provide information about the ductility of the material being tested and are also sometimes included on the tensile test certificate.

Round tensile testing is usually carried out in accordance with the testing standards ISO 6892-1, or ASTM E8 / E8M for room temperature testing. ISO 6892 Part 2 describes the method of test for elevated temperatures.

9.4.2 Cross-weld tensile tests

Cross-weld tensile tests are usually machined square or rectangular in cross section with a thickness equal to the full thickness of the welded plate (as seen in Figure 9.1). Sometimes the weld cap is ground off, but not always. The specimen grips are wider than the gauge containing the weld joint to avoid failure at the grip locations. Cross-weld tensile tests can be carried out to ISO 4136.

The principal difference between round tensile tests and cross-weld tensile tests is the intention of the test. Round specimen test data can be used quantitatively to determine the properties of specific materials and weld metals. Cross-weld tensile tests are used qualitatively to demonstrate that a weld metal has a higher strength than the parent metal. The gauge in a cross-weld tensile specimen contains the weld metal, as well as the heat-affected zones either side of the weld, and the parent metal. This is a mix of many microstructures with many different tensile properties. In this test all regions are, therefore, subjected to the same stress, and failure occurs in whichever microstructure has the lowest tensile strength. Each of these different microstructures also deforms by a different amount as yielding starts in each microstructure at different stages throughout the test before final rupture occurs. There is little purpose in determining the yield point for a cross-weld tensile test, because it does not correspond to the yield strength of any specific microstructure but is affected by a complex mix of all of them. The only information that should be reported from a cross-weld tensile test is the ultimate tensile strength of the specimen, and the location of failure (whether in the weld or parent metal). This can highlight the weakest part of the joint. It is generally desirable to have the weld metal strength overmatching that of the parent metal so that if the structure yields, the deformation does not become disastrously concentrated in the weld metal, but is spread throughout the parent material in the structure. An acceptable location of failure in a cross-weld tensile test is, therefore, usually in the parent metal.

9.4.3 Effect of tensile specimen orientation and locations

Within a rolled plate of steel, the microstructure in the rolling direction has grains which have been deformed and elongated, whereas transverse to the rolling direction there is no such elongation. These different microstructures give rise to different tensile properties in the longitudinal and transverse orientations relative to the plate-rolling direction and sometimes it is necessary to perform tests on specimens machined from different orientations to ensure these effects are measured.

In thick plate, it might also be necessary to determine the tensile properties in the through-thickness direction of the plate. Often this direction has the lowest ductility as a result of the rolling operation. However, in many welded structures loading is carried by the cross section of the plate and the through-thickness stresses are low so that the properties in this third direction (called the 'Z' direction in an X, Y, Z coordinate system) are not significant. However, some welded joint designs (such as tee butt welds or nozzles welded on pressure vessels) can result in through-thickness shrinkage and residual stresses occurring. If the steel has not been specified as a 'Z

grade' steel with guaranteed through thickness ductility, it may suffer from lamellar tearing after fabrication. For steel plate to be qualified as Z grade, a short transverse reduction in area (STRA) test, which is similar to a shortened tensile test, must have achieved at least a 20% reduction in area to be considered fully resistant to lamellar tearing in these joints. In order to make a tensile test specimen in this orientation, it is usually necessary to weld extension pieces to the top and bottom of the plate.

The location that the tensile specimen is machined from within a thick section component can also make a difference to the tensile properties that are measured. For example, a large casting cools much more slowly in the centre, giving a coarser grained microstructure and lower strength. It would be prudent to extract tensile test specimens from the centre (as well as nearer the surface) so that the lowest strength region is measured, and that may limit the allowable stresses.

9.5 Charpy testing

The Charpy test measures the energy absorbed when a notched bar of material is struck with a fixed-weight pendulum swung from a given height. The absorbed energy (in joules) is also referred to as the impact toughness of the material. However, it should be noted that impact toughness is rather different to fracture toughness, and Charpy data should not be used to carry out fracture mechanics calculations (see Chapters 5 and 11).

9.5.1 Test procedure

The standard Charpy-V notched specimen is 55 mm long and 10 mm square in cross section. Subsize specimens of 10 mm by 7.5 or 5 mm are also permitted, with the impact toughness requirements for them dependent on specimen size. Subsize Charpy specimens sometimes need to be tested at a lower temperature than standard sized specimens; the required temperature shift is given in the relevant fabrication code. On one face of the Charpy specimen there is a 2 mm deep machined notch which has a tip radius of 0.25 mm. The Charpy test specimen size is therefore independent of the component material thickness, and the notch does not try to replicate a sharp crack in any way. It acts only as a sufficient stress concentration to force the failure at that location in the specimen.

Charpy data for parent materials is sometimes provided by the material supplier. The specimen (and therefore the notch) orientation with respect to the plate rolling direction is significant. For rolled products, there can be a difference in impact properties between specimens notched parallel or transverse to the rolling direction. For welded joints, Charpy tests are often performed on specimens notched at the weld centreline, the fusion line, and the fusion line plus one or two millimetres, sometimes up to fusion line plus 5 mm, depending on the specification. For welds in plates greater than 25 mm thick, it might be necessary to extract Charpy specimens from near the weld cap, and from near the weld root as well.

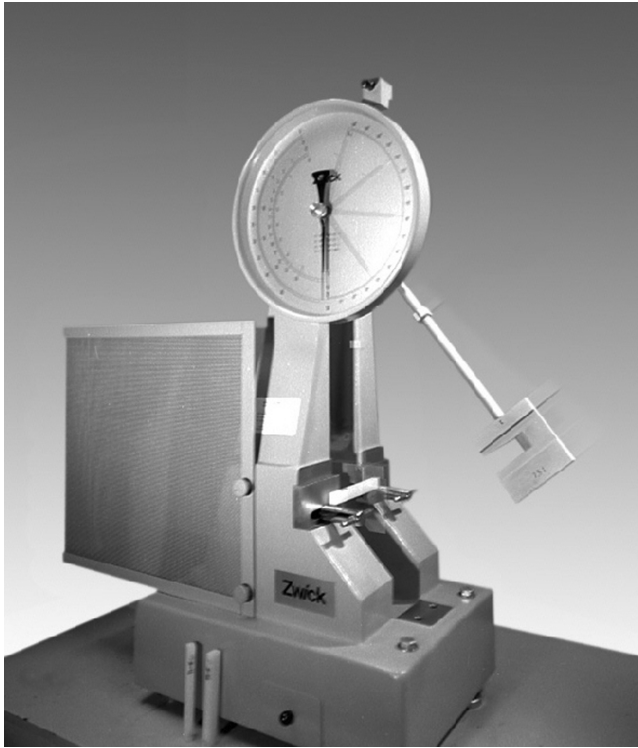


Figure 9.4 Charpy impact testing of a notched bar specimen placed horizontally across the test machine. The loading is applied by a weighted striker swung from a given height (© TWI).

The British Standard for Charpy testing is BS EN ISO 148-1 and the American Standard is ASTM E23. The standards differ only in the specific details of the strikers used. However, in both standards, the striker has a rigid arm and a weighted end, with the part of the striker that hits the back of the Charpy specimen having a narrower profile to concentrate the impact force. The striker is held up at the top of the machine, while the Charpy specimen is inserted. The test specimen may have been kept in a temperature-controlled bath for several minutes to get the specimen to the required test temperature. The specimen is quickly transferred to the test machine and put into position with the notch at the front. The pendulum is then released and the striker swings down and hits the back face of the specimen at the lowest point of the swing (Figure 9.4). The Charpy specimen breaks, and the amount of energy absorbed by the specimen during fracture reduces the speed of follow-through of the swing. The difference between the two heights at the beginning and end of the swing corresponds to the energy used in breaking the specimen, denoted as the absorbed energy in (J).

This absorbed energy when plotted against temperature generates a transition curve, as described in Section 5.2.2.

After the test, the failed Charpy specimens are examined and the percentage of shear area, corresponding to the amount of ductile failure, is quantified (see Chapter 8). Sometimes this is expressed as the percentage crystallinity which is equivalent to the amount of brittle failure. Percentage crystallinity is simply the inverse of percentage shear area, assuming that the Charpy specimen fractures purely in a mixture of brittle and ductile failure modes. A brittle material when Charpy tested makes a ‘ting’ sound and the striker swings up fairly high after the test. A ductile material sounds more like a ‘thud’, and the striker may swing only a short distance or be arrested completely by the test. The appearance of failed Charpy test specimens is also used to generate a ductile-to-brittle transition curve, as discussed in Chapter 8. During failure of the Charpy specimen, deformation can also occur in the lateral direction, squashed outwards on the compression side of the specimen. After the test the lateral expansion is measured, and expressed as a percentage increase in width from the original specimen size. The lateral expansion gives a further measure of the specimen’s ductility, and a minimum amount of expansion is sometimes a requirement for materials specifications.

9.5.2 Using Charpy data

Often three identical Charpy specimens are tested at the same temperature (some examples are shown in Figure 8.14). The temperature for the Charpy test is usually the minimum design temperature. Some fabrication codes impose a further reduction in the test temperature below the minimum design temperature in order to account for the differences between Charpy data and fracture toughness behaviour. The average of the three Charpy values at this temperature is required to meet a certain value given in the specification, and the minimum value must also exceed a given impact energy. The minimum value is based on many years’ experience that materials and welds satisfying this requirement are unlikely to experience brittle fracture in service. The minimum value depends on steel strength level and is typically 27 or 40J of absorbed energy. Specifying these values ensures that, in terms of Charpy toughness, the material is above its lower shelf at the minimum design temperature.

Charpy testing should be considered qualitative insofar as the results cannot be used to make calculations about a material’s fracture toughness behaviour. The results of a Charpy test are compared with previous Charpy data, usually given as a minimum Charpy requirement. Fortunately, the cheap and simple specimen design and test methodology for Charpy testing means that there is a large amount of Charpy data available, and more is produced every year. Where Charpy tests exceed the specification requirements for impact toughness, it can be confidently assumed that the material is unlikely to experience problems with brittle fracture. If, however, a Charpy test result is slightly less than the specified minimum value of 40J for instance, then it is not possible to quantify what effect this may have from the Charpy test. This is where fracture toughness testing is required.

9.6 Fracture toughness testing

The principle of the fracture toughness (or fracture mechanics) test is to test a material specimen containing a crack-sharp notch (Figure 9.5). Fracture toughness specimens are designed to be able to replicate the crack conditions of real flaws in metal structures. This allows the results of fracture toughness tests to be applied to actual structures and components to determine their tolerance to defects.

In Chapter 5, the principles of fracture mechanics are discussed. In fracture mechanics, the behaviour of flaws depends on local conditions at the crack tip. Fracture occurs when the crack driving force (the force that propagates the crack) just exceeds the material resistance to fracture (toughness). During the fracture toughness test, the load (crack driving force) is increased gradually until the crack starts to extend. Fracture toughness is calculated from the applied crack driving force

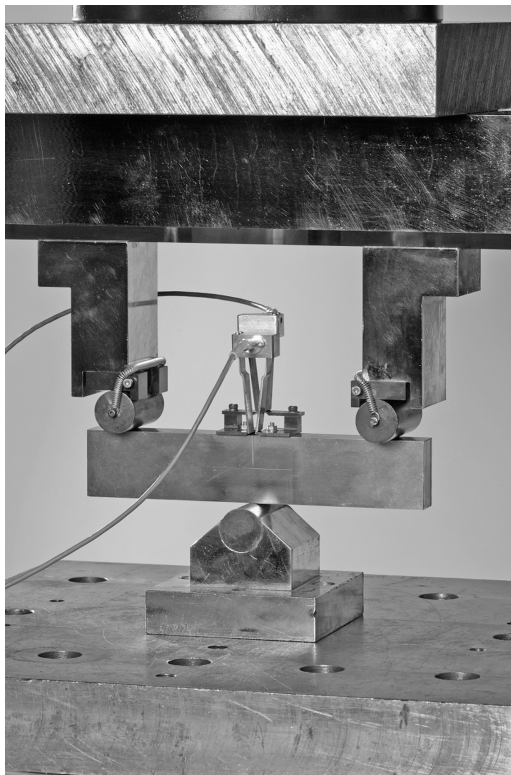


Figure 9.5 Fracture toughness testing (sometimes called CTOD testing), in which a specimen with a pre-cracked notch is tested under three-point loading until unstable fracture occurs. It is instrumented with a double-clip gauge across the crack mouth to measure the displacement associated with the increasing load (© TWI).

at the point of failure in the test. The factors that influence the value of fracture toughness include constraint, thickness, temperature and loading rate.

Both the crack driving force and fracture toughness can be characterised by one of several parameters: stress intensity K ; the crack tip opening displacement (CTOD), δ ; and the J integral. These are described further in Chapter 5. Any or all of these can be obtained from the same fracture toughness test. For brittle materials, K is the most appropriate fracture parameter. For ductile materials, it is better to use J , whereas CTOD can be applied to a wide range of fracture behaviour from brittle to ductile. In some industries, CTOD is the preferred fracture parameter to such an extent that fracture toughness tests are referred to simply as CTOD tests.

When deciding to carry out fracture toughness testing, there are several choices to be made. First, as fracture toughness depends to a great extent on microstructure, the target area to be tested must be selected, whether that is parent material, heat affected zone (HAZ) or weld metal. Next, the specimen size and geometry must be chosen, along with the notch orientation and depth.

9.6.1 Test specimens

The most common kind of fracture toughness test specimen is the single edge notched bend (SENB) specimen, as shown in Figure 9.5. Alternatives are the compact tension (CT) specimen, or the single edge notched tension (SENT) specimen. Fracture toughness specimens are usually full thickness, or as thick as possible when fully machined from the component material. As thickness increases, fracture toughness reduces, and so when the data is to be used for an engineering critical assessment (ECA; see Chapter 11), it is important to make sure the lowest (most conservative) value of fracture toughness is measured for a given component thickness. This means testing a specimen as thick as possible for the material in the structure.

For a SENB specimen, the dimensions, expressed as thickness $\times$ width, can either be square section ($B \times B$) or rectangular in cross section ($B \times 2B$), where B is the material thickness. The specimen length is machined to be $4.6W$ so that the test span is $4W$, where the specimen width W , is either equal to B or $2B$ depending on the cross section chosen. Therefore, unlike Charpy specimens, the specimen size in a fracture toughness test scales with the component material thickness.

The test is carried out using a typical ratio of crack depth (a) to specimen width (W) of around 0.5. Initially, a notch is made either by a slitting disc or electrodischarge machine. Low-stress cyclic loading is then applied to extend the notch a few millimetres by fatigue, called fatigue pre-cracking. A fatigue crack is the sharpest crack possible, enabling the results of the fracture toughness test to be analysed to fracture mechanics principles. However, the pre-cracking must achieve as straight a crack front as possible, and restrict the size of the plastic zone at the crack tip. If the fatigue pre-crack plastic zone size is excessively large it could affect the result of the test itself (plastic zones are discussed further in Chapter 5). The effectiveness of the pre-cracking can only really be confirmed after the fracture toughness test by breaking open the specimen to examine the shape of the fatigue pre-crack (Figure 9.6).

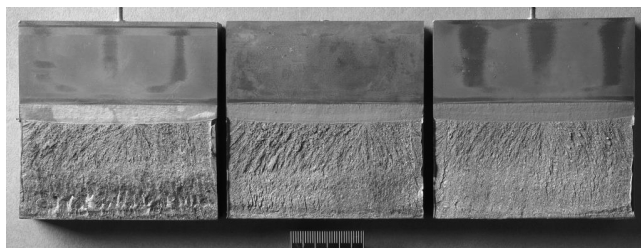


Figure 9.6 Appearance of fracture toughness test specimens after testing, showing the machined notch at the top, the fatigue pre-crack across the middle, and the brittle fracture during the test at the bottom of each specimen. The scale bar is in millimetres (© TWI).

If plain material specimens are to be tested, then the choice of specimen type and the orientation of the notching can be less important, but for specimens machined from welds, these differences in specimen design could give different values of fracture toughness. Welded joints are of particular importance when considering fracture, owing to the combination of residual stresses, stress concentrations, variations in microstructure and materials properties, and the likelihood of weld flaws. However, fracture toughness testing of weld and HAZ specimens is significantly more complicated than for parent material specimens. When a target microstructure is to be tested, it might be a single weld bead or a narrow region within the HAZ, the position of the crack tip is important to ensure that the target microstructure is adequately sampled. Welds are, by their nature, unlikely to have straight sides through the plate thickness, or to be perfectly aligned along the joint seam, so it can be almost impossible to be certain that a desired crack tip location has been achieved within the test specimen until it is broken open after the test and sectioned for post-test metallography.

9.6.2 Testing welds

Welds are also challenging for fracture toughness testing because it may be difficult to extract a suitable test specimen from a welded joint, owing to the joint geometry (such as fillet welds). Welding residual stresses may also affect the way in which fatigue pre-cracks grow and, subsequently, affect the shape of the resulting fatigue pre-crack. For weld specimens, some consideration must be made of how to reduce the effect of residual stresses on the fatigue pre-cracking, such as using local compression. Applying a local compression of up to 1% total strain across the ligament ahead of the notch location before pre-cracking can relieve some of the weld residual stresses and help to improve the shape of the fatigue pre-crack. This can make the difference between having a very bowed, or curved pre-crack front or achieving a much straighter and less curved pre-crack shape that gives more accurate test results.

When notching weld and HAZ specimens, there are two general approaches. Firstly, a ‘weld positional’ or WP notch is positioned with respect to a defined reference line in the weld such as the weld centre line, or crossing the fusion line at mid-thickness. The position of the notch, and hence the crack tip, is chosen in this case to sample all relevant microstructures. An example of a WP notch is a $B \times 2B$ SENB specimen through-thickness notched down the weld centreline. There is no requirement to carry out post-test metallography, and this approach is usually used to find a lower bound toughness because the notch samples a whole range of microstructures across its length, and the lowest toughness region is expected to initiate the fracture.

Secondly, a ‘specific microstructure’ or SM notch can be used to test a target microstructure, such as the grain-coarsened HAZ, or an individual weld bead. This might need to be a $B \times B$ surface notched SENB specimen surface notched to sample the fusion line at the crack tip. Post-test metallography is required to confirm whether the crack tip sampled the required region or microstructure. The uncertainty in ensuring the target notch location may mean that a larger number of tests are needed in order to generate the required number of ‘valid’ test results.

9.6.3 Test procedure

Before the fracture toughness test, the specimen is brought to the temperature of the test and left to soak at that temperature for a given time (depending on the size of the specimen). At low temperature, a bath of solvent is cooled with liquid nitrogen, or solid carbon dioxide, and, at high temperature, electrically heated pads can be used. When ready for testing, the specimen is slowly loaded under three-point bending. A number of parameters are measured during the fracture toughness test, including the applied load and loading rate (using load cells and displacement transducers in the servohydraulic test equipment), together with the temperature near the crack tip (using a thermocouple attached to the specimen). In addition, the crack-mouth opening is measured using clip-gauges attached to knife edges at either side of the crack mouth. A clip gauge comprises two cantilevered beams on which are positioned four strain gauges. By measuring the elastic strains that develop in these gauges during the test it is possible to determine the crack-mouth opening. It is possible to use one clip gauge, but by using two clip gauges (as shown in Figure 9.5) at two different heights above the crack mouth, CTOD can be established directly from the two clip gauge measurements, without making assumptions about the point of rotation, whereas two gauges can also be used to provide more data for determining J .

Load versus crack mouth displacement traces are generated during the test. The value of fracture toughness is determined from the point at which maximum load is reached on this graph, or at the point of fracture if maximum load is not reached. Ductile materials on the upper shelf show load-displacement traces that continue beyond maximum load (Figure 9.7), and the fracture toughness is given the subscript ‘m’, for example a J_m or a CTOD, δ_m . On the lower shelf, brittle materials do not

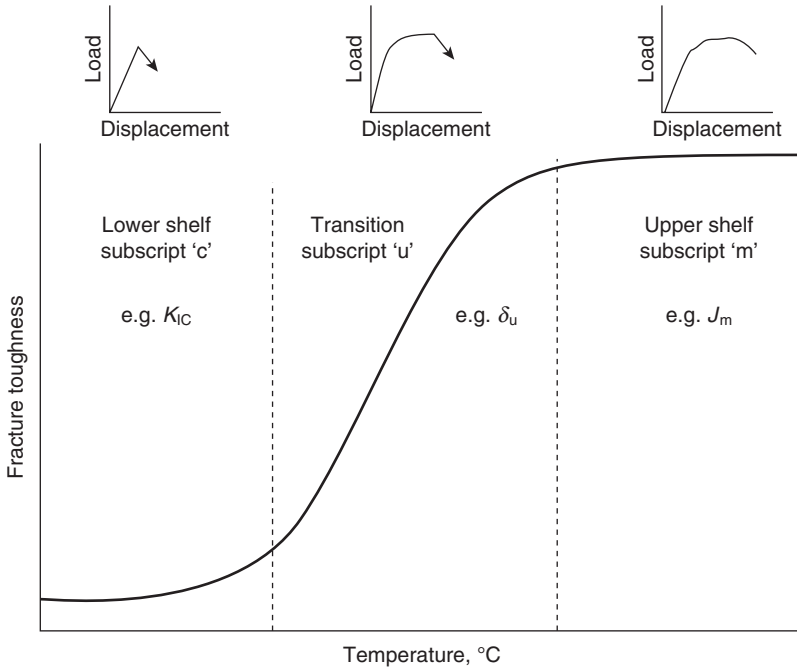


Figure 9.7 Sketches of typical load versus displacement traces from fracture toughness tests in the different regions of the ductile-to-brittle transition curve. The subscripts associated with each type of fracture behaviour are shown.

show any significant ductility before fracture occurs, and are given a subscript 'c', such as for a K_{IC} result. In the transition regime, there is some ductility (at least 0.2mm of stable crack extension measured on the fracture face), but not enough ductility to reach maximum load, this behaviour is noted with the subscript 'u', for example δ_u . The same fracture toughness test can be used to calculate a value of J , CTOD or K depending on which equations are used in the post-processing of the results.

For ductile materials, the ductility shown on the load versus crack mouth displacement trace is usually as a result of some stable crack extension, or tearing, in a ductile manner. As the crack extends, it deforms the plastic zone at the crack tip and absorbs energy, actually increasing the fracture toughness at the crack tip for relatively low levels of tearing. This improvement in fracture toughness with stable crack growth can be measured and accounted for in an ECA by determining a tearing resistance curve (also called an R-curve) instead of a single value of initiation fracture toughness (Figure 9.8). An R-curve is simply a plot of J or CTOD against the amount of stable tearing (up to about 1.5 mm), and usually shows a rising curve. The crack extension can be estimated by repeated partial unloading during the test and estimating the crack length from the unloading compliance (the slope of the unloading portion

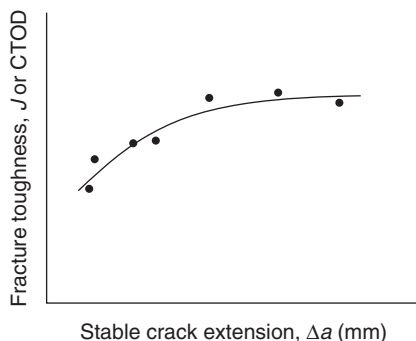


Figure 9.8 A fracture toughness tearing resistance curve (R-curve) fitted to a number of experimental data points over a range of stable crack extension.

of the graph). Alternatively, at least six identical specimens can be tested to different loading levels during testing and the crack extension measured after the test to plot an R-curve with the six data points.

Fracture toughness tests in plain material are carried out to a number of different standards of which BS 7448 Part 1, ISO 12135, and ASTM E1820 are the most common, the latter two giving guidance on single point and R-curve testing. For generating R-curves, BS7448 Part 4 is used instead of Part 1. For specimens notched into weld or HAZs, the testing standard to be used is BS EN ISO 15653, which gives additional guidance on pre and post-test metallography, notching and fatigue pre-cracking; this standard also relaxes the validity criteria compared with parent metal specimens.

When carrying out fracture toughness testing, usually a set of three identical specimens are tested, and the lowest value from the set of three is used for the ECA (see Chapter 11). When carrying out tests near the ductile to brittle transition temperature, there is an inherent amount of scatter in the test results and it may be necessary to perform a larger number of tests. It is also possible to perform fracture toughness tests over a range of temperatures to generate a ductile-to-brittle transition curve.

It is now obvious that a Charpy test (which involves a relatively blunt notched specimen loaded under high strain rate from the impact of the striker), and a fracture toughness test (which involves a sharp fatigue pre-cracked specimen slowly loaded under quasistatic conditions) are very different. The relative transition temperatures obtained from a ductile-to-brittle transition curve for Charpy specimens and fracture toughness specimens are also different. Although impact loading reduces the effective ductility measured in the sample, the smaller size of the Charpy specimen relative to a fracture toughness test specimen has the opposite effect. It is therefore not usually possible to quantify the exact ductile-to-brittle fracture toughness temperature from a Charpy ductile-to-brittle transition, but the transition regions are within a range of a dozen or so degrees of each other.

9.7 Fatigue testing

The mechanism of fatigue is discussed in chapter 6 and the classification of different weld joints into different fatigue classes depending on their stress concentration effect is given in Chapter 7. Using fatigue test data within a fitness-for-service assessment is discussed in Chapter 12. There is a significant amount of established fatigue data on welded joints in the public domain, such as the S–N curves for different weld joints in fatigue design standards such as BS 7608 (Guide to fatigue design and assessment of steel products). However, in order to characterise the fatigue performance of a particular joint, either in a specialised environment, or to qualify a joint for a particular application, it is sometimes necessary to carry out fatigue testing. These tests can usually be divided into small-scale fatigue tests and full-scale fatigue tests. Fatigue testing of weld specimens is not always carried out using standardised procedures, but rather in accordance with the needs of the individual test. However, some of the current test standards are given in 9.13.

To generate an S–N curve for a particular joint, it is necessary to test a number of identical test pieces (typically six to ten). The test piece is clamped in the fatigue test machine and subjected to cycles of a given stress range (Figure 9.9). Each test piece

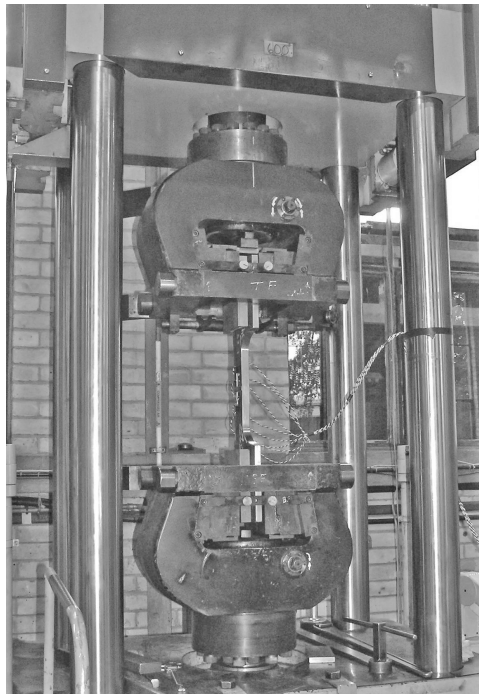


Figure 9.9 A steel specimen with welded attachments under axial constant amplitude fatigue testing to determine the fatigue joint performance of this configuration (© TWI).

is tested at a different stress range until the test piece breaks in two. The number of cycles at failure is recorded; this is often many thousands of cycles. By plotting the data (typically using a log–log scale) from each of the test specimens as a number of cycles to failure (N) for a given stress range (S), it is possible to plot an S–N curve, as described in Chapter 6. Fatigue test machines for small-scale tests such as these can apply tensile, compressive or alternating stress ranges. It is possible to test specimens under axial loads (as shown in Figure 9.9) or under bending or torsion. Modern fatigue testing machines use servohydraulic controls to apply a fluctuating load, which can either be constant amplitude (in order to generate data to plot a S–N curve), or variable amplitude, in order to test a joint under known fluctuating loading conditions.

Sometimes the weld joint to be assessed is too large to be tested in a standard fatigue testing machine, for example a pipeline girth weld. A full-scale fatigue test on a pipeline girth weld (such as would experience variable loading as an offshore riser pipe) is now commonly performed using resonance fatigue testing (Figure 9.10). Resonance fatigue testing exploits the natural frequency, or resonant frequency, of the pipe under bending to generate a rotating bending stress along the length of the pipe. The ability to apply a large number of stress cycles in a relatively short time makes the test an efficient way to evaluate the fatigue performance of pipe joints, and it can be applied to diameters from 100 to 1000 mm. Resonance fatigue testing is becoming an increasingly popular method for weld procedure qualification of pipe joints that are expected to undergo significant fatigue loading during their lifetime.



Figure 9.10 Resonance fatigue testing of pipeline girth welds. Full-scale test specimens are cyclically loaded using an eccentric load at one end until resonant frequency is achieved, such that a large number of stress cycles can be imposed on the pipe in a short time (© TWI).

Tests can be performed either with a zero mean stress by applying the resonance load alone, or at a given mean stress.

Fatigue crack growth rates in many materials can be measured using similar specimens to those used for fracture toughness tests. However, the environment that a component experiences in service alters (often increases) the fatigue crack growth rate. This effect has been examined for marine environments, both with and without applied cathodic protection systems. However, for structures which experience fatigue loading in more challenging environmental conditions (such as 'sour' hydrogen sulphide service during oil and gas production), it may be necessary to perform mechanical testing to determine the fatigue crack growth rate in these environmental conditions. Fatigue crack growth rate tests use notched specimens that are cyclically loaded under bending while immersed in the required environment. Corrosion fatigue tests are conducted at the same frequency as the anticipated service loading frequency to allow enough time for the corrosive environment to interact with the material at the crack tip. For structures subject to wave loading, the relevant frequency is typically 0.2 Hz.

Fatigue crack growth rate tests can be performed to ASTM E647 or BS ISO 12108, whereas corrosion fatigue tests are covered by BS EN ISO 11782-2.

9.8 Creep testing

The creep test is designed to test a material's resistance to deformation at high temperature. Creep is a mechanism through which material under a significant load (but below the yield strength) experiences deformation after a long time at high temperature. This can ultimately lead to void formation, necking and finally rupture. For creep to be a concern, the service temperature needs to be a significant proportion of the metal's melting temperature to provide enough thermal energy for the atomic diffusion that is necessary for creep to occur. Some alloys such as lead suffer creep at ambient temperatures (which explains the sagging of lead water pipes, or the slumping of medieval lead glazing at the bottom of stained glass windows over time owing to gravity). For welded engineering materials, creep is mainly a concern for components operating at the high temperatures of power generation plants.

The creep test involves holding a metal specimen, in design very similar to a tensile test specimen (but with tighter machining tolerances for straightness and surface finish), under a constant deadweight load at high temperature for a long time. The constant load is applied simply by suspending weights from the specimen via a lever arm (seen in Figure 9.11). The test specimen is surrounded by a tubular furnace held by a frame around the specimen. The furnace is thermostatically controlled to maintain a constant temperature to within 2 or 3 °C. The temperature is measured using a thermocouple attached to the test specimen. The load and temperature are assumed to be constant during the test, and the extension of the gauge and time are measured. A very sensitive extensometer is positioned across the gauge to record any increase in length over the duration of the test; this might only be a few per cent by the end of the test. The strain versus time graph can then be plotted.



Figure 9.11 Creep testing rig, under a deadweight load during calibration. During creep testing the heating component cylinder is brought down to surround the test specimen seen in the middle of the picture (© TWI).

This creep curve shows three distinct phases as the creep life progresses. The primary creep stage shows an accelerating rate of strain owing to work hardening. However, this then settles into a steady rate of strain over time, known as secondary creep. This comprises the majority of the creep life of the specimen. As the deformation of the specimen becomes significant, voids form, necking occurs and the strain rate accelerates again until final failure. This is the tertiary stage of creep.

The results of creep tests allow components to be designed well into the secondary stage of creep, but avoiding the tertiary stage. The creep life might be based on a certain amount of strain, perhaps 2% strain over a lifetime of 90 000 h at 450 °C, which could be determined from the creep curve.

An alternative creep test is the creep rupture test, which, instead of monitoring the extension of the specimen throughout the test, simply applies a given load at

a temperature and records the time until failure. The same equipment is used, but this simpler variant can be used to set design lives for components where the dimensional changes are less significant, and only the load-bearing capacity over time is important. Creep testing is carried out to either BS EN ISO 204:2009 or ASTM E139.

9.9 Corrosion testing

Corrosion testing can take many forms, but, in principle, it involves exposing a material to a chemical environment under controlled conditions and measuring any cracking or corrosive damage, such as weight loss that occurs as a result. Corrosion occurs with many different combinations of alloy and chemical environment, so there is no universal corrosion test suitable for all materials. The corrosion risk may be pitting, for example with duplex stainless steels in chloride service, or stress corrosion cracking at welds, for example carbon and low-alloy steels in wet sulphide-containing (sour) environments (Figure 9.12). Testing may be performed on plain bars of material immersed in a corrosive fluid, or the specimens may also be under an applied load and/or at elevated temperature to accelerate the corrosion reactions.

A simple accelerated test technique for assessing the pitting corrosion resistance of duplex stainless steel welds in chloride-containing environments is given in ASTM G48. The test involves total immersion of weld test specimen bars in an aqueous solution of ferric chloride at a constant given temperature for 24 h. It assesses whether a specimen has passed or failed based on the weight loss measurement.

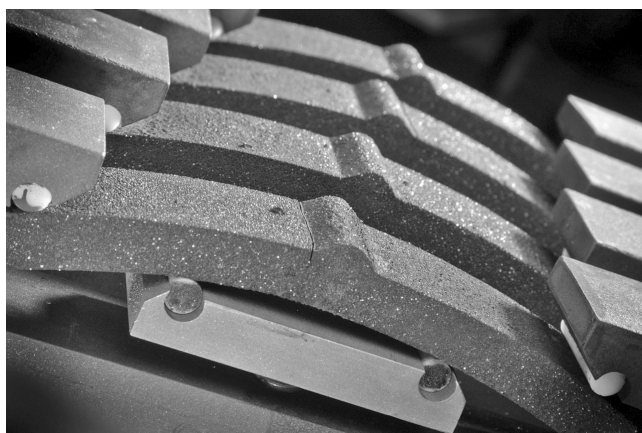


Figure 9.12 Heavily rusted welded specimens after corrosion testing by immersion in a sour environment under four-point-bend loading. The nearest specimen shows a sulphide stress corrosion crack along the weld toe (© TWI).

Corrosion tests in general are not standardised, because the individual test is carried out to replicate actual conditions which might be experienced rather than a standard environment. However, general guidance to suitable methods is given in ASTM NACE / ASTM G31 *Standard Guide for Laboratory Immersion Corrosion Testing of Metals*.

9.10 Macrographic sections

A macrographic section taken from the weld is often called simply a 'macro'. It is a full thickness slice through the weld, polished and etched to reveal the shape and microstructure of the weld.

The macro sample's surface is first ground flat using a series of successively finer wet grit silicon carbide papers. The subsequent polishing is carried out using fine particle diamond paste on a cloth-covered polishing wheel. The polishing removes all the grinding scratches, and the macro specimen has a mirror finish at this stage. Steels are then usually etched in 2% nitric acid (nital) solution, stainless steels can be etched in 20% sulphuric acid, whereas non-ferrous alloys require other specialist etchants to reveal their microstructures. The suitable etchant for each material is an acid that dissolves each of the different components of the microstructure at a different rate, revealing the grain boundaries and other microstructural constituents in contrast against the main background phase. The etchant is swabbed over the specimen, and rinsed off using a solvent after sufficient time to reveal the microstructure. This may be done in several short etching stages to avoid overetching the specimen and, thus, making the microstructure less clear.

A macro section is a very useful qualitative test to show the placement of each individual weld bead in a multipass weld (Figure 9.13). The differences in microstructures mean that the parent metal, HAZ, and the weld metal appear significantly different on the macro of a typical arc weld and can be clearly seen. Welding defects such as pores, cracks, lack-of-fusion or lack of penetration are clear on the macro when examined at up to ten times magnification.

Examining the specimen under higher magnification between 100 and 1200 times is called 'micro' rather than 'macro' examination. At these scales, it is possible to determine the microstructures in the weld and HAZ, to identify inclusions, and quantify the relative amounts of microstructural constituents.

9.11 Hardness testing

Hardness testing assesses a material's resistance to indentation. The shape and size of the indenter differs between the most common methods, but the general principle is to apply a known force to an indenter of a given shape onto the material's surface. The size of the indentation that is left is then related to a measurement of hardness. Hardness testing requires a flat, often ground and polished surface, which may be a

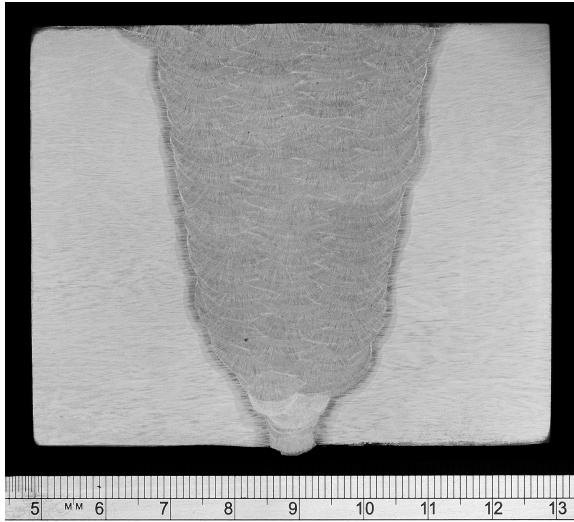


Figure 9.13 'Macro' of a multipass butt welding in steel, etched in alcoholic 2% nitric acid solution (nital) to show the weld passes, HAZ and parent metal in the joint. The root passes in this weld were deposited using a different weld consumable, and appear contrasted with subsequent weld fill passes (© TWI).

weld macro section etched so that the different regions of the weld can be identified and the hardness indents located in the correct region. The most common hardness testing methods are Vickers, Rockwell and Brinell hardness testing, used for general testing of bulk materials and quality control during processing.

9.11.1 Vickers hardness test

In the UK and Europe, it is most common to use Vickers hardness testing (Figure 9.14). This is one of the oldest hardness testing methods, and has a wide hardness scale, making it suitable for most metals and welds. The Vickers hardness test uses a 136° pyramidal diamond indenter that forms a square indent. The load is applied for a time of 10–15 s. The two axes of the diamond shaped indentation measured in millimetres are averaged (to give the dimension d) and the hardness is determined, based on a calibration for different kilogram loads (P). The hardness (HV) is expressed as, e.g., HV10, for a 10 kg load, or HV5 for a 5 kg load.

$$HV = 1.8544 \frac{P}{d^2} \quad (9.1)$$

Vickers hardness testing is carried out to standards such as ASTM E384 or ISO 6507 which has four parts describing the test itself, calibration and hardness tables.



Figure 9.14 Vickers hardness testing machine (© TWI).

9.11.2 Brinell hardness test

An alternative method is the Brinell hardness test, which uses a hardened steel (or tungsten carbide) ball indenter with a diameter D of, usually, 10 mm. This is applied under a load P of 500–3000 kg applied for 10–30 s. The diameter of the circular indentation d is measured in millimetres. The hardness number, (HB) is calculated using the following equation:

$$\text{HB} = \frac{P}{\frac{\pi D}{2} (D - \sqrt{D^2 - d^2})} \quad (9.2)$$

The Brinell hardness testing standard is ASTM E10. For both Vickers and Brinell hardness testing, it is common to use reference tables to give the value of hardness for a given indentation size and load, instead of making a calculation each time.

9.11.3 Rockwell hardness test

The third kind of test, mostly favoured in the USA, is the Rockwell hardness test. There are a number of variations of the Rockwell hardness test called 'scales' that use different indenters and loads. For example 'HRC' is the hardness tested using the Rockwell 'C' scale, and 60HRC is not the same hardness as 60HRB. The choice of scale depends on the hardness of the material to be tested; softer alloys such as aluminium alloys are assessed using the B scale, whereas steels and harder alloys are more suited to the C scale.

The general principle of the Rockwell hardness test in all cases is to compare the depth of indentation under a low preload with the subsequent indentation under a full load. The value of Rockwell hardness HR is given by $(E-e)$, where E is a constant (130 for a steel ball indenter and 100 for a diamond cone indenter), and e is the permanent increase in penetration depth arising from the main load (divided by 0.002 mm). Readings above 100HR for a ball indenter are rejected and a different scale is used to obtain sufficient accuracy in the reading. A diamond cone indenter is not used for hardness of 20HR or less, because of the risk of damaging the indenter in these cases.

Rockwell hardness testing can be carried out to either ISO 6508-1 or ASTM E 18. Conversion tables are available to convert between Vickers, Brinell and Rockwell hardness values, such as those given in ASTM E140.

9.11.4 Microhardness testing

The three methods described in the previous subsections give good general hardness unaffected by microstructural variability. However, if a weld has a very narrow HAZ, if hardness testing of individual grains is required, or if a very small component needs to be hardness tested, it is possible to use microhardness testing as an alternative. Microhardness testing is carried out using either laboratory equipment, or portable methods that allow materials to be tested in service or on site.

A bench-based Vickers microhardness test machine (Figure 9.15) is similar in principle to a standard Vickers hardness tester, except much lower loads are used (between 1 and 1000 g) and, therefore, much smaller indents are formed into the specific region of interest. A similar principle is used in the Knoop hardness test, except that an elongated pyramid indenter is used.

An alternative microhardness method is the ultrasonic hardness test, which does not use the size of an indentation to determine hardness. Instead, a Vickers diamond attached to the end of a metal rod is vibrated at its resonant frequency by a piezoelectric converter. When this is brought into contact with the specimen the resonant frequency is changed. The size of the impression produced by a small applied load and the change in frequency are used to determine a hardness value. The very small size of the indentation means that this type of hardness test is considered effectively non-destructive.

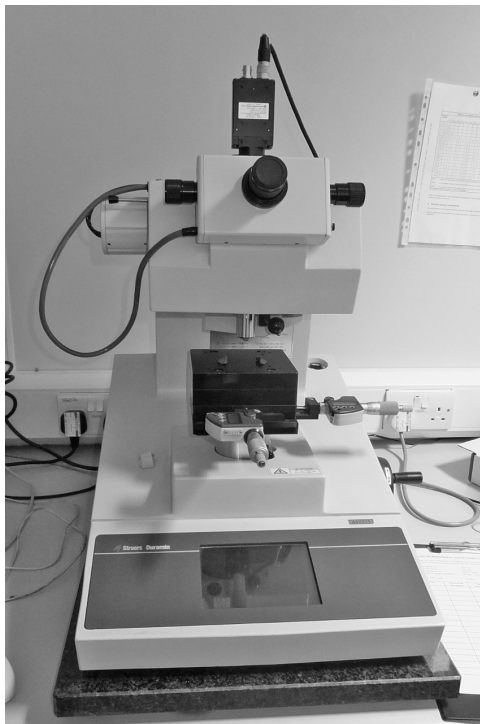


Figure 9.15 Microhardness testing machine (© TWI).

9.11.5 Portable hardness testing

The ultrasonic hardness testing principle is also used in portable hardness testing equipment. This is used to test large components, as a quality control method, or taken to site to measure the hardness of structures in service, for example to verify whether a correct postweld heat treatment has been carried out. There are several commercially available portable hardness testers, such as the one shown in Figure 9.16. This type of equipment can be selected to give hardness in any type of units through its electronic programming, either Vickers, Brinell or Rockwell.

Other kinds of portable hardness testers are based on the Rockwell, Brinell or Vickers hardness testing principles. Some portable hardness testers involve the dynamic rebound of a spring-loaded ball bearing to determine hardness. The effectiveness of any kind of hardness equipment is in the regular calibration of its hardness measurements against a block of material with a known hardness value. Good quality hardness data is also dependent on having well trained operators of the equipment and well prepared surfaces to test. This is true of all hardness testing, but



Figure 9.16 Microdur portable surface hardness testing equipment supplied by GE Inspection Technologies (© TWI).

is particularly important for portable hardness equipment where surfaces are prepared in the field, unlikely to be flat, and the calibration is required more frequently.

9.11.6 Using hardness data

Hardness values can be used to estimate the ultimate tensile strength of carbon steel (by multiplying the Vickers hardness by 3.3). This can be a quick and simple way of obtaining sufficient data for quality control of the material being supplied to check that it meets a certain strength level or range of tensile properties. For welds, control of hardness is important to ensure that hydrogen cracking or sulphide stress corrosion cracking does not occur. If the weld has a hardness less than 380HV, the formation of hard brittle microstructures that are susceptible to hydrogen cracking such as martensite is unlikely. A hardness of up to 380HV is often the maximum allowed for weld procedure qualification, in order to ensure hydrogen cracks are avoided. The risk of sulphide stress corrosion cracking occurs at even lower hardness and, for this reason, a limit of 248HV is imposed for the root of welds likely to contain sour products.

9.12 Conclusions

Carrying out an extensive set of mechanical tests on a welded joint may seem a costly and time-consuming exercise. However, every weld is a unique combination of the

parent material, welding parameters and weld consumable, thus forming a weld with its own range of properties. The only way of gaining confidence in knowing the properties of that joint is to perform the appropriate mechanical tests.

This chapter has covered most of the common types of mechanical testing that are performed on welds and other metal specimens in order to calculate their fitness-for-service. The tests fall into three main categories.

First, there are the mechanical tests that are commonly used for weld procedure qualification, including bend tests, cross-weld tensile tests, Charpys, macros and hardness tests. Their main purpose is for quality control and validation of the welding procedure. The results are pass or fail, based on a given permissible limit in the test. A 'fail' might require modification of the weld procedure and/or retesting.

Second, there are the mechanical tests that are performed to generate data for design and engineering calculations, such as tensile and fracture toughness tests. The tensile and fracture toughness data are used in ECA or weld-strength calculations. The results of the tests are not dependent on reaching a required minimum value, but low strength or low fracture toughness might result in the calculation of lower design stresses or smaller flaws being acceptable during fabrication.

Third, there are the more specialised tests that are intended to mimic particularly challenging aspects of the expected service conditions, such as cyclic loading in fatigue tests, high-temperature service in creep tests or a particular chemical environment in corrosion tests. The results of these types of tests directly relate to the performance of the joint in service and can give an indication of its susceptibility to these types of damage mechanisms and, therefore, how to avoid premature failure of the structure.

9.13 Sources of further information and advice

- ASME BPVC (2013) *Section IX – welding, brazing, and fusing qualifications*, American Society of Mechanical Engineers.
- ASTM E8 / E8M-11 (2011) *Standard test methods for tension testing of metallic materials*, American Society for the Testing of Materials.
- ASTM E10-12 (2012) *Standard test method for Brinell hardness of metallic materials*, American Society for the Testing of Materials.
- ASTM E18-11e1 (2011) *Standard test methods for Rockwell hardness of metallic materials*, American Society for the Testing of Materials.
- ASTM E23-12c (2012) *Standard test methods for notched bar impact testing of metallic materials*, American Society for the Testing of Materials.
- ASTM E139 (2011) *Standard test methods for conducting creep, creep-rupture, and stress-rupture tests of metallic materials*, American Society for the Testing of Materials.
- ASTM E140-12be1 (2012) *Standard hardness conversion tables for metals relationship among Brinell hardness, Vickers hardness, Rockwell hardness, superficial hardness, Knoop hardness, scleroscope hardness, and Leeb hardness*, American Society for the Testing of Materials.
- ASTM E190-92 (2008) *Standard test method for guided bend test for ductility of welds*, American Society for the Testing of Materials.

- ASTM E384-11e1 (2011) *Standard test method for Knoop and Vickers hardness of materials*, American Society for the Testing of Materials.
- ASTM E606 / E606M-12 (2012) *Standard test method for strain controlled fatigue testing*, American Society for the Testing of Materials.
- ASTM E647-13E1 (2013) *Standard test method for measurement of fatigue crack growth rates*, American Society for the Testing of Materials.
- ASTM E1820 (2013) *Standard test method for measurement of fracture toughness*, American Society for the Testing of Materials.
- ASTM G48 (2011) *Standard test methods for pitting and crevice corrosion resistance of stainless steels and related alloys by use of ferric chloride solution*, American Society for the Testing of Materials.
- ASTM NACE / ASTM G31-12 (2012) *Standard guide for laboratory immersion corrosion testing of metals*, American Society for the Testing of Materials.
- ASTM TP91A (1963) *A guide for fatigue testing and the statistical analysis of fatigue data*, American Society for Testing and Materials.
- BS 7270 (2006) *Metallic materials. Constant amplitude strain controlled axial fatigue. Method of test*, British Standards Institution.
- BS 7448-1 (1991) *Fracture mechanics toughness tests – method for determination of K_{Ic} , critical CTOD and critical J values of metallic materials*, British Standards Institution.
- BS 7448-4 (1997) *Fracture mechanics toughness tests – method for determination of fracture resistance curves and initiation values for stable crack extension in metallic materials*, British Standards Institution.
- BS 7608 (2014) *Guide to fatigue design and assessment of steel products*, British Standards Institution.
- BS EN ISO 148-1 (2012) *Metallic materials. Charpy pendulum impact test – test method*, British Standards Institution.
- BS EN ISO 204 (2009) *Metallic materials. Uniaxial creep testing in tension. Method of test*, British Standards Institution.
- BS EN ISO 4136 (2012) *Destructive tests on welds in metallic materials – transverse tensile test*, British Standards Institution.
- BS EN ISO 6507 (2005) *Metallic materials – Vickers hardness test*, British Standards Institution.
- BS EN ISO 6892-1 (2009) *Metallic materials – tensile testing. Part 1: Method of test at room temperature*, British Standards Institution.
- BS EN ISO 6892-2 (2011) *Metallic materials – tensile testing. Part 2: Method of test at elevated temperature*, British Standards Institution.
- BS EN ISO 11782-2 (2008) *Corrosion of metals and alloys. Corrosion fatigue testing. Crack propagation testing using precracked specimens*, British Standards Institution.
- BS EN ISO 15614 (2004) *Specification and qualification of welding procedures for metallic materials*, British Standards Institution.
- BS EN ISO 15653 (2010) *Metallic materials. Method of test for the determination of quasistatic fracture toughness of welds*, British Standards Institution.
- BS ISO 1143 (2010) *Metallic materials. Rotating bar bending fatigue testing*, British Standards Institution.
- BS ISO 1352 (2011) *Metallic materials. Torque-controlled fatigue testing*, British Standards Institution.
- BS ISO 12108 (2012) *Metallic materials. Fatigue testing. Fatigue crack growth method*, British Standards Institution.
- BS ISO 12110 (2013) *Metallic materials. Fatigue testing. Variable amplitude fatigue testing*, British Standards Institution.

- BS ISO 12111 (2011) *Metallic materials. Fatigue testing. Strain-controlled thermo-mechanical fatigue testing method*, British Standards Institution.
- Chandler, H. (1999) *Hardness testing, 2nd revised edition*, ASM International, ISBN-10: 0871706407, ISBN-13: 978-0871706409.
- ISO 6508-1 (2005) *Metallic materials – Rockwell hardness test. Part 1: Test method (scales A, B, C, D, E, F, G, H, K, N, T)*, American Society for the Testing of Materials.
- ISO 12135 (2002) *Metallic materials – unified method of test for the determination of quasistatic fracture toughness*, International Standards Organisation.
- Lee, Y-L., Pan, J., Hathaway, R. and Barkey, M. E. (2004) *Fatigue testing and analysis: theory and practice*, Butterworth–Heinemann Ltd, ISBN-10: 0750677198, ISBN-13: 978-0750677196.
- Nayar, A. (2005) *Testing of metals*, McGraw–Hill Professional, ISBN-10 070581649, ISBN13 9780070581647.
- Verink, E. D. (1993) *Corrosion testing made easy: the basics*, National Association of Corrosion, ISBN-10: 1877914622, ISBN-13: 978-1877914621.

10.1 Introduction

The presence of defects in a weld is a major influence on the risk of brittle fracture or fatigue failure. However, in order to assess how susceptible a given weld is, it is important to be able to detect any defects present in that weld and to assess their size.

In the early days of welding development, the methods available to detect defects were fairly crude, and although surface-breaking flaws could be found, buried defects posed a greater challenge. However, over the past two decades there have been significant leaps forward in the development of non-destructive testing (NDT) methods, which follow a few common principles. Advances in electronics and signal processing have greatly increased detection capability and enabled the sizes of flaws to be determined much more accurately.

Non-destructive testing methods are used to detect imperfections in a material without destroying or damaging the component itself. The main advantage of non-destructive testing, compared with destructive mechanical testing (as outlined in Chapter 9), is that it allows examination of many or all of the welds after fabrication without affecting those welds. It is possible to identify defects buried deep inside the material, and structures that are already in service can be inspected (without damaging them), which is an important part of ensuring the continuous structural integrity of the structure throughout its lifetime.

10.2 'Perfect' welds and detection of weld defects

Despite the general population's dislike of the idea that anything safety critical might be inherently 'flawed', this is the situation that we have to accept when considering welds. There is no such thing as a 'perfect' weld that contains no defect or imperfection. This is directly analogous to saying there is no such thing as a perfect material; every crystalline material contains lattice defects such as dislocations and vacancies. The different types of common fabrication defects are discussed in Chapter 3. Even with the best welding procedure performed by the most skilled welder, it is likely that there are tiny imperfections in the weld. In most as-deposited welds, the weld toes contain microscopic intrusions that are too small to detect from the surface, but

sufficient to initiate fatigue cracks. In a multipass weld, it is not uncommon to find small, sporadic lack-of-fusion flaws between the weld passes or at the side wall of the weld. These small imperfections are often collectively considered as meeting 'workmanship' quality, i.e. the size of defects that would be reasonably expected from normal welding workmanship.

On occasion, however, flaws larger than those typical of good workmanship are generated, often as a result of deficiencies in the welding procedure. This might be because during welding the right conditions were present for hydrogen cracking to occur, or in a mechanised weld, because the parameters were wrongly set and a lack-of-fusion defect can extend along a substantial length of the weld before being rectified. For any type of defect, it is necessary to find it, and to assess how big it is. Only then can decisions be made as to whether repair is necessary, often by determining whether the joint and structure are fit-for-purpose (see Chapter 11).

Not all welds are inspected along their entire length; for example it might not really matter if all the welds in a gate are not inspected and the occasional flaw passes through the net, as the consequences of failure are fairly low. However, a very different approach is taken when fabricating a high pressure steam boiler. If it is decided that only a proportion of the entire length of a weld is to be inspected, then sometimes an additional safety factor (also known as a joint factor in some codes) is applied to reduce the allowable stress.

Given that perfect welds do not exist, then the detection of weld defects goes hand in hand with the quality systems associated with welding. Welding inspection and NDT are therefore vital to both in ensuring weld integrity and avoiding failure.

There are many inspection and NDT methods available, and often more than one is used when inspecting a weld. Some are better at detecting the locations of flaws, but are less able to assess the size of a flaw; some methods detect only surface flaws, whereas other methods are better suited to finding embedded flaws. A suite of NDT methods is therefore necessary to ensure that all types of flaw are detected adequately.

10.3 Visual inspection

The first step in NDT is to perform a visual inspection. At its simplest this involves looking at the appearance of the weld, usually with additional lighting and magnification in order to do a thorough job. Sometimes a photographic record is kept of any indications found. Visual inspection is able to highlight issues such as spatter (Figure 10.1) which occurs if the arc is unstable. The shape of the ripple on the weld surface shows the direction of travel during welding, because the curves on the ripple follow the back edge of the weld pool as it progresses (visible in Figure 10.1). Visual inspection also detects any weld shape imperfections such as misalignment, lack of root penetration or excessive root penetration, underfill, or an excess of weld metal forming a peaky weld. These defects are not usually important for weld integrity but they can indicate where a weld procedure has not been followed closely or that the weld procedure is not optimised. Both are symptoms of deficiencies in welding quality systems that may give rise to more serious weld defects.

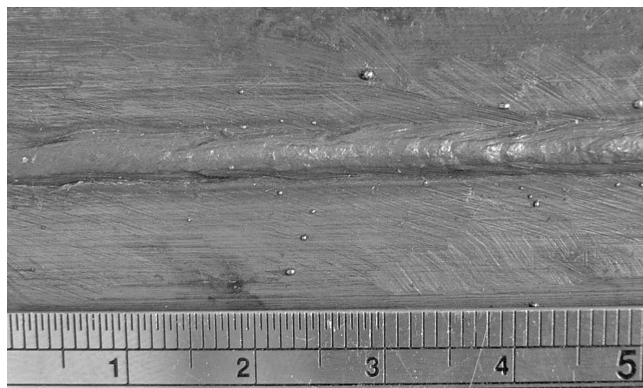


Figure 10.1 Surface appearance of a weld, showing some spatter, and a fairly peaky surface profile. The welding direction is left to right, as shown by the ripple shape (© TWI).

The various welding processes produce welds of differing appearances. The welding inspector uses clues from the visual appearance of the weld when checking that the correct procedures have been followed. The quality of a visual inspection depends on having access to the weld (for example if it is inside a pressure vessel) and on having the weld well illuminated. Sometimes borescopes or remote cameras are used inside small cavities and the images are fed back to a computer screen for interpretation. Footage or still images can be kept as a permanent record of the inspection. Visual inspection is usually carried out in accordance with BS EN ISO 17637.

10.4 Dye penetrant inspection

A simple technique to enhance the visual appearance of surface-breaking cracks is to make them easier to detect by showing them up in greater contrast against a pale background. Two techniques are available: dye penetrant (sometimes called liquid penetrant) testing, and magnetic particle inspection (MPI). Penetrant testing (PT) has the advantage that it can be used on any material, whereas MPI is restricted to magnetic materials.

In penetrant testing, a dye is applied to the weld and seeps into any cracks present on the surface by capillary action. Excess dye is removed, a white developer is sprayed on, and the dye inside the crack ‘bleeds’ back into the developer, thus showing up the crack in strong contrast (Figure 10.2). The penetrant is a dark dye (often deep red) or a fluorescent dye which is viewed in a dark room under ultraviolet light; fluorescent dyes are often more sensitive than standard dyes.

Penetrant testing although fairly simple, requires skill to correctly apply. First, the weld surface is cleaned of any spatter or slag that could absorb the penetrant dye and mask any cracks present. A degreasing cleaner is used on the surface to remove any



Figure 10.2 Chevron-shaped surface-breaking cracks detected in weld metal using dye-penetrant testing. The dye is usually dark red, exposed onto a background of a white developer to highlight any cracks found (© TWI).

mill scale or grease that could hinder the capillary action of the penetrant. This cleaner is usually sprayed on from an aerosol and wiped off using a cloth, leaving the surface to dry. The penetrant (also an aerosol) is sprayed over the weld area and, after about 15 min (to allow the dye to seep into any surface-breaking flaws) any remaining penetrant is cleaned off using a cloth. The cleaner is not sprayed on again to avoid washing out any dye inside the cracks. Finally, the developer is sprayed on from a third aerosol (giving rise to the name the 'three can system'). The developer forms a white powdery chalk-like layer that contrasts with the dye that it absorbs from any crack by reverse capillary action, thus allowing the cracks to be identified and the length measured. Good lighting is required to obtain optimum results from the method and allow photographs to be taken as a record. All the chemicals are water-based, but after the inspection both the dye and the developer must be cleaned off the component again.

Dye penetrant testing is a simple inspection method that may be used on any type of material or shape of component. It allows a large surface area to be inspected and the materials needed to perform the inspection are inexpensive. However, it does require thorough surface preparation and only detects surface-breaking defects. Penetrant testing standards include BS EN ISO 3452; Part 1 describes the general principles, Part 2 covers the penetrant testing, Part 3 is about the reference test blocks, and Part 4 covers the equipment required. Flaw acceptance levels are given in BS EN ISO 23277.



Figure 10.3 Magnetic particle inspection (MPI): a suspension of magnetic particles sprayed onto a weld, to which a magnetic field is being applied from a permanent magnet. The weld has previously been painted white in order to show up any defects (© TWI).

10.5 Magnetic particle inspection (MPI)

For ferromagnetic materials (ferrous alloys such as iron and steel), MPI is an alternative to penetrant testing. In MPI, a magnetic field is induced in the material by use of a permanent magnet or electromagnet. Any defects in the weld cause interruptions in the magnetic flux field, and these are made visible when a powder or liquid suspension of tiny magnetic particles is sprayed on (Figure 10.3). The magnetic particles are attracted to the edges of the defect where there is a discontinuity in the magnetic field; effectively, the defect acts like a magnet itself.

Instead of using a permanent magnet, the magnetic field can be induced electromagnetically. Probes are applied to the surface of the component and a large current is passed through to generate the magnetic field. The magnetic particles may be black or red, or, for greater sensitivity fluorescent magnetic particles, which are viewed under ultraviolet light, are used. Magnetic powders are particularly useful when the component is hot, and the solvents could cause a fire hazard. As in penetrant testing, a good level of light is necessary to view the component, and photographs are taken to record any defects found. MPI allows detection of some slightly subsurface defects as well as surface-breaking ones. The method is portable when permanent magnets are used. MPI can be used to inspect components with a complex shape, and even works underwater!

There are some limitations to MPI, however. The method works only on ferromagnetic alloys and does require reasonable surface preparation. The magnet must be applied in two perpendicular orientations to make sure that all the defects are detected, because a magnetic field in the same orientation as a crack is not disrupted and does not show up the flaw. After the inspection, it is sometimes necessary to demagnetise the component to remove any permanent magnetic field

resulting from the application of magnetic fields. MPI is carried out to BS EN ISO 17638 for welds and to BS EN ISO 9934 Parts 1 to 3 for parent materials. Acceptance levels for flaws in welds are given by BS EN ISO 23278.

10.6 Eddy-current testing

The principle of electromagnetic induction is that an alternating electric current induces an associated magnetic field and, conversely, a fluctuating magnetic field induces an electric current. Eddy-current testing involves a probe containing a small wire coil through which an alternating current is passed, thus inducing a magnetic field around the probe. When the probe is brought into close contact with a metal surface, the magnetic field induces circular currents in the metal, centred around the probe location. These are called eddy currents (analogous to the small eddies of whirlpool turbulence in water). The eddy currents are themselves associated with smaller magnetic fields, which can be detected either by the same probe coil or a secondary coil. Defects in the material cause disturbances to the magnetic fields around the eddy currents so that monitoring the eddy current magnetic fields allows defects at the surface and just below the surface to be found. Changes in both the amplitude of the magnetic field and the phase of the changing fields can provide information about any flaws detected in the component.

The depth of penetration of the eddy currents determines the depth of defects which can be detected. The depth to which the eddy currents penetrate into the material is inversely related to the frequency of the probe coil current (which ranges from a few Hertz to a few megaHertz) and to the magnetic permeability of the material and its electrical conductivity. Ferromagnetic materials such as iron naturally have a high permeability, which varies with changes in composition or microstructure (particularly around welds). These large variations in magnetic permeability can make the signal response difficult to interpret. High-frequency probes are used to detect skin effects and near surface conditions. Low-frequency probes are used to inspect subsurface regions.

Although the technique requires significant operator skill, the complex interactions of the electromagnetic fields in eddy-current testing mean that direct interpretation of the signal is not possible (Figure 10.4). To obtain information about flaw geometry and position, the probe is calibrated on a test block machined with slots of a known size (half or one millimetre depth for instance). The signal at the plate edge and that for the probe lifting off the surface are calibrated. The signal from defects in the calibration block can be compared with any signals detected during the inspection. It is possible to design eddy current probes for particular applications. The probe coils may be wrapped around a component such as a tube or round bar, as close to the surface as possible. Alternatively, the coil can be mounted on a probe that can go inside a tube, for example part of a tube-sheet array where external access is not possible. The quality of the inspection in these instances depends on the calibration blocks and the reference holes and slots that have been machined in them. Eddy-current testing is an inspection technique that relies heavily on calibration and



Figure 10.4 Eddy current testing of a geared wheel component (© TWI).

reference blocks to set the sensitivity of the signal before every scan. However, just because the probe gives a similar signal to that from a 1 mm slot in a calibration block does not mean that the defect detected is 1 mm deep. The comparison is usually qualitative only, because difference in the defect angle, width and surface roughness all affect the signal quality. Flaws oriented parallel to the plate surface are not detected at all because they do not interrupt the induced eddy current parallel to the surface.

Eddy-current testing can be used on painted surfaces to detect cracks beneath the paint or other coatings. As well as inspecting welds for defects, it can also be used to compare magnetic and electrical conductivity of materials for quality control purposes. Eddy-current testing cannot be used on complex geometries, nor does it give a permanent record of the inspection. Because Eddy-current testing cannot be used to assess size of defects it is often used as a quality control or screening method simply to detect the presence of defects. Eddy-current testing can be carried out in accordance with BS EN ISO 15549 and BS EN ISO 15548-1.

10.7 Radiography

Anyone who has broken a bone and gone to hospital for an x-ray is familiar with the use of radiography to detect defects hidden within structures (Figure 10.5). In an engineering context, radiography detects defects buried within metals and welds, using either a gamma-ray source or an X-ray generator. The component is exposed to the radiation for a given time, during which a photographic film or a digital detector is held behind the component. The more radiation that passes onto the film, the darker the resulting image. The welded component obstructs the rays and appears lighter. Any wall thinning, porosity or cracks in the metal allows more radiation to pass, and shows up darker against the paler component. Excess weld metal or root penetration shows up even paler as it impedes the rays even more.



Figure 10.5 Radiograph image being examined over a light box. This image is of a riveted aircraft component (© TWI).

To create the radiographic image, the film is prepared with identification marks using letters made from lead (which show up bright white against the black film). The sensitivity of the image and the size of the smallest indication that can be resolved are measured using an image quality indicator (IQI), which is included in each image. There are two main kinds of IQI: the wire type and the step hole. A step-hole IQI is a piece of material with steps of successively thicker material. Holes of different sizes are drilled in each step. The radiographic sensitivity is the diameter of the smallest visible hole divided by the thickness of the test material, expressed as a percentage. Wire type IQIs comprise a standard set of six or seven wires of different diameters, and the radiographic sensitivity is the diameter of the smallest visible wire as a percentage of the test material thickness. The IQI is placed on the component at the side of the weld or region of interest, in a location of the greatest thickness, where the least sharpness is expected. The sizes of any defects found when the radiograph is examined, are related to the sensitivity of the IQI, and, thus the size of any flaw that could be missed is estimated.

In radiography, the choice of radiation source depends on the application. An X-ray source is produced by a beam of electrons from an incandescent filament accelerated by a voltage to strike a tungsten target that then generates X-rays, which are guided out through a special window towards the component. The current through the filament controls the amount of radiation produced, and the voltage between the filament and target controls the energy of the radiation, i.e. its penetrating power. The X-ray generating equipment requires a power source so it cannot be used for inspections on site, but it can be safely turned on and off when required.

An alternative is to use a gamma ray source consisting of a piece of radioactive material. The source is stored within a shielded container and only temporarily taken out to make the exposure for the radiography. Gamma-ray sources are small, portable

and robust, and require no external power, making them a better choice than X-ray sources for site applications, or for inspecting welds which would otherwise be inaccessible. They also offer greater penetrating power than conventional X-ray sources and there is less scatter in the beam. However, the exposure times required are longer for gamma radiography than for X-ray techniques. The images from gamma-ray sources have less contrast and are grainier than those using X-rays. Also, it is necessary to replace the gamma-ray source when the radiation eventually decays.

All radiographic sources are damaging to human health, particularly gamma-radiation sources, which cannot be 'turned off'. If radiography is being carried out on site or in a workplace, it is important to clear the area of any personnel and set up safety barriers before work commences. This makes radiography time-consuming and disruptive to other production processes nearby. Personnel who work with radioactivity regularly have to take a set of additional measures to ensure that their exposure to radiation does not exceed recommended levels.

The general principles of radiographic testing are covered by BS EN ISO 5579. Radiography methods are covered in BS EN ISO 17636, Part 1 covers methods using film, whereas Part 2 covers digital detectors. Radiography of aluminium welds is covered by BS EN ISO 10675-2.

10.8 Ultrasonic testing (UT)

As for radiography, many people are familiar with the use of ultrasound in medical applications to 'see' inside something in order to detect what might otherwise have been hidden. Ultrasound scans are routine during pregnancy to image the unborn foetus, and the principle when applying ultrasonic inspection to engineering structures is similar.

A probe that produces high frequency (ultrasonic) sound waves is held against the surface, which is covered with a suitable gel couplant so that as much of the ultrasound as possible passes into the component. The probe emits a pulse of ultrasound, which is reflected and diffracted by any discontinuity or defect in its path. A detector records these reflected ultrasound waves (the 'pulse echo') and this is used to generate an internal image of the structure. A pulse which enters straight down into defect-free metal is shown in the detector as a large response from the pulse at the top surface of the plate from waves which did not couple with the material, and another large response from waves reflected from the back surface of the plate. The time delay between these pulses depends on the thickness of the material (based on the speed of sound in that particular material). In this way, ultrasonic testing (UT) is used to measure wall thickness for the assessment of corrosion wall thinning (Figure 10.6). However, if there are internal defects in the material, they are detected as smaller signal responses from a distance somewhere between the front wall and back wall signals, and the location of the top and bottom of the defect relative to the material surface can be identified.

The frequency of the pulse is chosen to optimise the penetrating power and the resolving power of the signal. High frequencies of 6–10 MHz have low penetrating



Figure 10.6 Ultrasonic probe being used to scan a component to quantify the extent of wall thinning owing to internal corrosion (© TWI).

power but give high resolution (and are used for aerospace applications), low frequencies of 0.5–2.5 MHz give high penetration but low resolving power (which can be used to test castings). A suitable frequency for inspecting welds lies between these two. It can be difficult to inspect very thin materials accurately using UT techniques.

As well as sending a pulse at 90° to the plate surface (called a ‘compression’ wave or longitudinal wave), the pulse can be sent into the material from a wedge-shaped probe at an angle, so that the waves become ‘shear’ or transverse waves. Shear wave angle probes are useful for inspecting welds where the shape of the joint requires the sound beam to be directed in at a certain angle in order to sample the required material, such as a weld side wall. A standard portable set of UT equipment contains probes for 0°, 45°, 60° and 70°, at frequencies of 2.5 and 5 MHz. The probes contain a piezoelectric transducer to produce the ultrasound, and to detect the response. The probes are connected to a box which contains the electronics for the signal processing and display (Figure 10.7). Ultrasonic test equipment requires a power source, but it is relatively simple and robust and can be operated in most environments. Calibration blocks are also necessary to set the sensitivity for UT.

The main advantages of UT are that it is very sensitive to planar defects when angle probes are used, and it can determine the depth of defects better than other NDT methods. Access is only required from one side, and the inspection is quite rapid and provides real time information. The disadvantages of manual UT are that it does not provide a permanent record, and the interpretation of the signal requires significant skill and training. There is some error on the flaw sizing measurements, and it is unsuited to coarse grained materials such as austenitic stainless steel. The

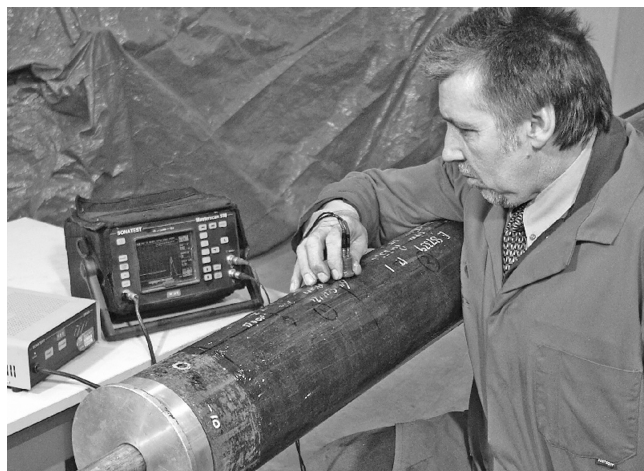


Figure 10.7 Manual ultrasonic testing of a pipe using a compression probe at an angle of 0° connected to an electronic display unit (© TWI).

coupling can also be challenging on rough surfaces, so some surface preparation is required.

Manual UT of welds in metallic materials thicker than 8 mm is carried out to BS EN ISO 17640, which has four testing levels labelled as A, B, C and D, each corresponding to a different probability of detection of flaws. For level D, specialist UT techniques are required for extremely high or low temperatures, or non-ferrous alloys. This standard can be used in conjunctions with BS EN ISO 11666 to provide suitable acceptance levels.

An alternative standard is BS EN 583, comprising Part 1 covering general principles, Part 2 on sensitivity and range setting, Part 3 on transmission technique, Part 4 on how to examine flaws perpendicular to the surface, Part 5 on characterization and sizing of discontinuities, and Part 6 specifically on the time-of-flight diffraction (TOFD) technique for detection and sizing of discontinuities. The equipment used for UT can be standardised to BS EN 12668, in three parts separately covering the instruments, probes, and combined equipment.

The ongoing development into ultrasonic probe design and controls, and signal processing capability, has led to a number of advanced UT methods, such as TOFD, phased-array, and long-range UT. Use of these methods increases the rate of scanning for a given area, increases automation and improves the accuracy of the flaw sizing capability.

10.9 Probability of detection

Different NDT methods have their respective strengths and weaknesses. One measure of the accuracy of an inspection method is the probability of detection of a given flaw

size. Some NDT methods can detect almost all the flaws present in a weld, but can not size them accurately, other methods can give accurate flaw sizes, but only if the operator knows where to apply the probe.

The probability of detection is used to determine statistically, the size of the largest flaw which might just be missed by the inspection technique. In a fitness-for-service assessment, it is important to be confident that no flaws greater than a specific size exist in a structure.

For example, conventional UT reliably finds surface flaws larger than $3\text{ mm} \times 15\text{ mm}$ within a weld of 10–25 mm thick, whereas a focused phased-array of ultrasonic probes could find flaws greater than $1.5\text{ mm} \times 10\text{ mm}$. Radiography can reliably detect volumetric flaws such as porosity greater than 1.2 mm in diameter. Surface detection methods such as penetrant testing or MPI can find flaws larger than $1.5\text{ mm} \times 5\text{ mm}$ on a machined surface, but on an as-welded plate with a poor weld profile, can only find defects larger than $4\text{ mm} \times 20\text{ mm}$. Advice on typical limits of detection are given in BS 7910 Annex T.

10.10 Flaw-sizing error

Once a flaw has been detected, the size of the flaw is then determined. The ability to detect a flaw, and the ability to size it may be completely independent. The error in the flaw length of a surface flaw detected using ultrasonic methods is typically $\pm 7\text{ mm}$ or even $\pm 10\text{ mm}$ for manual UT. A better flaw length accuracy is given by eddy-current testing or penetrant testing which can give $\pm 5\text{ mm}$, whereas radiography can be accurate to $\pm 2\text{ mm}$ in flaw length. However, the flaw height measurement is usually the most important for assessing integrity and MPI, PT and radiography are not able to measure this dimension. In contrast, ultrasonic methods are able to measure flaw height. Manual UT can size a flaw height to $\pm 3\text{ mm}$, whereas phased-array UT can improve the size accuracy to $\pm 1.5\text{ mm}$. These typical flaw-sizing errors are based on those given in BS 7910 Annex T.

It is important to recognise that all inspection methods have limitations, and that the dimension(s) ascribed to flaws that have been detected during the inspection may not accurately reflect the size of the flaw that is actually present. This has important consequences when making assumptions for a fitness for service assessment (see Chapter 11). However, improvements in NDT method design, automation, signal processing, and operator skill can increase the accuracy of the inspection significantly.

10.11 Choosing suitable non-destructive testing (NDT) methods

Once the principles of the available NDT methods are understood, then the choice of which NDT methods are best suited to particular applications can be made, and the typical error in the flaw sizes reported.

The objective of the inspection clearly influences the choice of inspection method to a great extent. If the objective is to detect a certain type of damage then one method might be the obvious choice, for example radiography is well suited to detect porosity, whereas MPI is used to detect fatigue cracks at weld toes. Often the inspection is being performed to provide a general overview of the flaws present, such as lack-of-fusion flaws along a weld seam. The flaws need to be detected and sized, and might be present as both surface and buried defects. In this instance, more than one method might be employed to complete the full inspection. It is usual for visual inspection to always be carried out before applying further NDT techniques.

If inspection is required to find defects of a particular type and orientation, then the chosen technique depends on the likely orientation of the cracks. Fatigue cracks are perpendicular to the applied loading on the structure, hydrogen cracks in pipeline welds may be transverse or longitudinal to the weld depending on the circumstances. The likely position of the expected flaws should also be considered before the inspection, i.e. whether the defects are localised at the weld (for example stress corrosion cracking under the influence of weld residual stress), or whether damage is more widespread (such as wall thinning owing to general corrosion). The most likely defects and degradation can be established using knowledge based on any previous failures or cracking, and from an understanding of the product and service of the component, i.e. (whether corrosion, fatigue or stress corrosion cracking is likely to occur).

The material to be inspected also determines the choice of method; UT methods are less suitable to large grained microstructures such as austenitic stainless steel, while MPI is only possible for ferromagnetic alloys. The thickness of the material also has an effect. For example, thick materials can be inspected by ultrasonic techniques that are not capable of detecting flaws in thin material. Eddy-current testing can be tuned to a penetration depth suitable for even very thin materials.

The ease of access to the material, whether it is in service or part of a large or complex fabrication, limits the choice of inspection method. Methods requiring their own power source pose a greater challenge for the inspection of inaccessible regions, even if their detection capabilities are most suitable. The condition of the surface to be inspected also needs to be considered, because UT, MPI and penetrant testing have greater accuracy with a clean and prepared surface; the conditions inside a large storage tank or pressure vessel are likely to be very different. Eddy-current testing and radiography are not so restricted by the surface condition. Additional surface preparation time has implications for the cost and time for inspection.

If it is necessary to keep a record of the results of the inspection, this might be more of a challenge for eddy current tests or UT, which provide no intrinsic record of the inspection, compared with radiography or penetrant testing where a photographic record is straightforward.

All of these considerations must be coupled with an understanding of the accuracy of the flaw sizing for each method:

- Visual inspection is vital to provide a first impression of the appearance of the weld, allowing any warning signs of poor weld quality to be identified, and the appropriate NDT techniques to be chosen.

- Penetrant testing can be used to determine the length of surface flaws but it does not give a measure of the flaw depth. The accuracy of sizing depends on the quality of the surface finish.
- MPI can provide flaw length for surface and, sometimes, near-surface flaws, but no assessment of the flaw depth is possible. Surface finish affects the quality of the inspection.
- Eddy-current testing can be used to determine the length of surface and slightly subsurface flaws. The surface finish does not matter, and eddy-current testing can be used to measure the length of surface flaws on painted or coated surfaces. An indication of the flaw depth can be determined by amplitude measurement and/or phase measurement, but the techniques for critical sizing are subjective, and are based on comparisons to responses from calibration blocks.
- Radiography is best for detecting volumetric defects, which it can accurately size and locate in plan view. However, the two-dimensional image produced gives no quantitative measure of the defect depth, or its through-thickness position unless further images are taken at different angles. Radiography poses significant safety hazards from the radiation source.
- UT allows the length and depth of surface and embedded flaws to be determined, as well as the through-wall position of the buried flaws. The accuracy of manual UT can be variable, but automated methods based on ultrasonic principles (such as TOFD or phased array methods) can improve the accuracy of the flaw height sizing. UT requires highly skilled operators and good surface preparation.

10.12 Conclusions

It is possible to get an impression of the weld quality by simply looking at it during visual inspection. There is no such thing as a 'perfect' weld, but quantitative measurements of any defects present are essential if the significance of those flaws is to be determined. It is possible to enhance the ability to find any surface-breaking defect by using penetrant testing or MPI. However, many potentially damaging welding defects are buried below the plate surface, making it necessary to choose the most suitable method to find the type of damage that is of most concern. There are different advantages and disadvantages for each of eddy-current testing, radiography or UT. There is no 'perfect' inspection method which can find and size all potential weld flaws. Some understanding of the type of flaws likely to be present and what information is needed about those flaws guides the choice of NDT method(s) employed. Weld inspection is a specialist field and has a symbiotic relationship with structural integrity.

10.13 Sources of further information and advice

- BS 7910 (2013) *Guide to methods for assessing the acceptability of flaws in metallic structures*, British Standards Institution.
- BS EN 583-1 (1999) *Non-destructive testing – Ultrasonic examination. General principles*, British Standards Institution.
- BS EN 583-2 (2001) *Non-destructive testing – Ultrasonic examination. Sensitivity and range setting*, British Standards Institution.

- BS EN 583-3 (1997) *Non-destructive testing – Ultrasonic examination. Transmission technique*, British Standards Institution.
- BS EN 583-4 (2002) *Non-destructive testing – Ultrasonic examination. Examination for discontinuities perpendicular to the surface*, British Standards Institution.
- BS EN 583-5 (2001) *Non-destructive testing – Ultrasonic examination. Characterization and sizing of discontinuities*, British Standards Institution.
- BS EN 583-6 (2008) *Non-destructive testing – Ultrasonic examination. Time-of-flight diffraction technique as a method for detection and sizing of discontinuities*, British Standards Institution.
- BS EN 1330-1 (1998) *Non-destructive testing – Terminology. List of general terms*, British Standards Institution.
- BS EN 1330-2 (1998) *Non-destructive testing – Terminology. Terms common to the non-destructive testing methods*, British Standards Institution.
- BS EN 1330-3 (1997) *Non-destructive testing – Terminology. Terms used in industrial radiographic testing*, British Standards Institution.
- BS EN 1330-4 (2010) *Non-destructive testing – Terminology. Terms used in ultrasonic testing*, British Standards Institution.
- BS EN 1330-5 (1998) *Non-destructive testing – Terminology. Terms used in eddy current testing*, British Standards Institution.
- BS EN 12668-1 (2010) *Non-destructive testing – Characterization and verification of ultrasonic examination equipment. Instruments*, British Standards Institution.
- BS EN 12668-2 (2010) *Non-destructive testing – Characterization and verification of ultrasonic examination equipment. Probes*, British Standards Institution.
- BS EN 12668-3 (2013) *Non-destructive testing – Characterization and verification of ultrasonic examination equipment. Combined equipment*, British Standards Institution.
- BS EN ISO 3452-1 (2013) *Non-destructive testing – Penetrant testing. General principles*, British Standards Institution.
- BS EN ISO 3452-2 (2006) *Non-destructive testing – Penetrant testing. Testing of penetrant materials*, British Standards Institution.
- BS EN ISO 3452-3 (2013) *Non-destructive testing – Penetrant testing. Reference test blocks*, British Standards Institution.
- BS EN ISO 3452-4 (1999) *Non-destructive testing – Penetrant testing. Equipment*, British Standards Institution.
- BS EN ISO 5579 (2013) *Non-destructive testing. Radiographic testing of metallic materials using film and X- or gamma rays. Basic rules*, British Standards Institution.
- BS EN ISO 9934-1 (2001) *Non-destructive testing – Magnetic particle testing. General principles*, British Standards Institution.
- BS EN ISO 9934-2 (2002) *Non-destructive testing – Magnetic particle testing. Detection media*, British Standards Institution.
- BS EN ISO 9934-3 (2013) *Non-destructive testing – Magnetic particle testing. Equipment*, British Standards Institution.
- BS EN ISO 10675-2 (2013) *Non-destructive testing of welds. Acceptance levels for radiographic testing – Aluminium and its alloys*, British Standards Institution.
- BS EN ISO 11666 (2010) *Non-destructive testing of welds – Ultrasonic testing – Acceptance levels*, British Standards Institution.
- BS EN ISO 15548-1 (2013) *Non-destructive testing – Equipment for eddy current examination – Instrument characteristics and verification*, British Standards Institution.
- BS EN ISO 15549 (2010) *Non-destructive testing – Eddy current testing – General principles*, British Standards Institution.

- BS EN ISO 17636-1 (2013) *Non-destructive testing of welds – Radiographic testing – X- and gamma-ray techniques with film*, British Standards Institution.
- BS EN ISO 17636-2 (2013) *'Non-destructive testing of welds – Radiographic testing – X- and gamma-ray techniques with digital detectors'*, British Standards Institution, 2013.
- BS EN ISO 17637 (2011) *Non-destructive testing of welds – Visual testing of fusion-welded joints*, British Standards Institution.
- BS EN ISO 17638 (2009) *Non-destructive testing of welds – Magnetic particle testing*, British Standards Institution.
- BS EN ISO 17640 (2010) *Techniques, testing levels and assessment for non-destructive and ultrasonic testing of welds*, British Standards Institution.
- BS EN ISO 23277 (2009) *Non-destructive testing of welds – Penetrant testing of welds – Acceptance levels*, British Standards Institution.
- BS EN ISO 23278 (2009) *Non-destructive testing of welds – Magnetic particle testing of welds – Acceptance levels*, British Standards Institution.
- Mix, P. E. (2005) *Introduction to nondestructive testing: a training guide*, 2nd edition, Wiley-Blackwell, ISBN-10: 0471420298, ISBN-13: 978-0471420293.
- Prakash, R. (2009) *Non-destructive testing techniques*, New Academic Science Ltd, ISBN-10: 1906574065, ISBN-13: 978-1906574062.

11.1 Introduction

When welds are designed, the joints are assumed to be sound. However, in practice, there is no such thing as a completely defect-free weld. Microscopic defects at the weld toes, called intrusions, are present in most welds. Flaws such as cracks, welding defects or corrosion damage can occur during fabrication or in service. It is, therefore, necessary to assess whether these flaws affect the ability of the structure to perform its required function, i.e. whether the weld is fit-for-service.

For many years, there was a general consensus that welds should meet a certain level of ‘workmanship’ that cracks should not be tolerated, and lack-of-fusion flaws and weld-shape imperfections should be limited. Although striving for the best weld quality is always an admirable intention, these limitations on flaw sizes were set with fairly arbitrary acceptable levels, based on the engineering experience of what ‘feels right’ for a weld. In the early days of welding codes, inspection methods were crude and very small buried defects were probably not detected and, where cracks were found, they were often sufficiently large to cause concern and be repaired. However, in recent years non-destructive testing (NDT) techniques have become increasingly sensitive, and it is now possible to detect and size flaws of only a millimetre or so in length and height (as described in Chapter 10). The question now arises of whether finding these defects is reason enough to reject the weld. Some criteria are required to decide whether the flaws are acceptable or not. A large flaw in a ship’s superstructure in a region of low stress might be completely irrelevant to the safe operation of the ship, however a tiny weld flaw in the shell of a highly stressed pressure vessel might be enough to cause the vessel to explode. At any point in the life of the structure, from design to fabrication and into operation, it may be necessary to decide whether the weld is fit to meet its current service requirements, or those it is likely to encounter in the future. Therefore, the assessment of each defect should be carried out based on the fitness-for-service of the weld in question containing that flaw.

11.2 Fitness-for-service assessment

In order to avoid failure and repairs, a fitness-for-service assessment is carried out.

11.2.1 *Avoiding failure*

Most high-integrity welding is carried out in accordance with certain codes and standards, depending on the particular welding application. The intention of these fabrication codes and standards is to ensure that, by following their set of rules, the structure is safe and the quality of the welding is high. Stresses are kept well below yield, and stress concentrations are avoided. The parent metal and welds are required to be subjected to a suite of mechanical testing to ensure they possess adequate properties including strength and toughness. Finally, there is a limit on the permissible flaw sizes, based upon workmanship principles. The requirements have been developed to err on the side of caution, and experience has shown that following the code requirements does indeed mean that the structure is unlikely to fail under normal conditions. The implication, however, is that any flaws larger than the acceptance limit of the fabrication code must be repaired.

11.2.2 *Avoiding repairs*

Weld repairs are very costly, up to ten times the cost of an original production weld. Repairs can halt production of other components and cause delays to project delivery. It may also be necessary to approve a new repair welding procedure in order to carry out repairs. During the repair, it often requires greater welder skill to re-weld manually a small region of a weld seam than in the original fabrication. Most importantly in terms of fracture and fatigue, repairs carried out on post-weld heat-treated (PWHT) steel can result in high levels of residual stress and even introduce further defects as a result of the repair. Performing weld repairs when it is not strictly necessary is not only expensive, but could be considered a dangerous thing to do with respect to the weld's integrity.

11.2.3 *Fitness-for-service principle*

The fitness-for-service approach to deciding whether flaws are acceptable is based upon making a strength and fracture mechanics based calculation for that flaw under its given loading conditions. The fracture mechanics description of the behaviour of cracks in structures is explained in Chapter 5. Fitness-for-service (FFS) assessment is also sometimes called engineering critical assessment (ECA). The procedures for carrying out the assessment usually follow an established code or standard, in order to have confidence that the results of the assessment ensure the structure is safe for its expected service life conditions. The fitness-for-service assessment calculates the likelihood of failure owing to ductile overload (also called plastic collapse), and the likelihood of failure owing to brittle fracture. It is also possible to consider the possible subcritical extension of the flaw by fatigue crack growth in the calculations (see Chapter 12).

A flaw is considered to be acceptable provided that the conditions for failure are not reached within the required service life.

11.3 When to carry out an engineering critical assessment (ECA)

An ECA can be carried out at a number of stages throughout the life of a structure. The purpose of the ECA might be very different at these different stages. For example, during design, an ECA can help to determine what material properties are required for the construction specification, or to validate design stress against postulated materials properties and potential defects. At this stage, the assessment uses assumptions about the material data and flaw sizes, and the intention is to explore the sensitivity of these factors on the design requirements.

More commonly, an ECA is performed before fabrication, based on materials properties data obtained by testing a sample weld made using the appropriate welding procedure. The assessment assumes the maximum design stresses and is used to determine flaw acceptance criteria and the subsequent inspection strategy. An alternative reason for an ECA at fabrication is to assess whether it is possible to avoid PWHT or a proof test, while still ensuring that the high integrity weld is still acceptable. If it were possible to show that these treatments were unnecessary it would reduce the time and cost of fabrication.

Once the weld is in service there may be further reasons why an ECA might be necessary. If flaws are detected during a routine outage inspection, a decision needs to be made as to whether to continue to keep the equipment offline and perform a repair, or whether it can return to service, either indefinitely, or until a planned repair can be scheduled. Carrying out an ECA helps in these decisions. It is useful at this stage if all the required input data, particularly the mechanical properties, have been established during weld procedure qualification testing. If fracture toughness data are needed for welds in service, the options are:

- a) to cut out a sacrificial piece of material for testing and weld in a patch;
- b) to fabricate an equivalent piece of weld using the same welding procedure; or
- c) to make conservative estimates of the toughness based on published or Charpy data.

If the flaw can be shown to be fit for continued service, the ECA might also provide information to help set suitable inspection periods to monitor the flaw for any further expected growth, for example by fatigue.

It is sometimes useful to perform ECA calculations during a failure investigation in order to assess candidate failure mechanisms. For example, an investigation was carried out on a large casting which had failed by brittle fracture. When the fracture surface was examined, a large casting defect was visible on the fracture surface. An initial theory for the cause of failure was that the defect should not have been there, and that it was the cause of failure by initiating the brittle fracture. The mechanical properties of the casualty material were obtained by testing and ECA calculations were performed. The ECA showed that in order for a flaw of the size measured on the fracture surface to cause the failure, the stresses on the component would have needed to be twice the level of the maximum allowable stress, which was not initially considered feasible or realistic under normal conditions. From the results of the ECA

calculations, the failure investigation then looked for reasons why an overload might have occurred, such as safety valves not operating correctly, or additional loads being experienced that were not accounted for in the design. After considerable further study, an additional unforeseen source of loading was identified that was sufficient to cause failure. Modifications were then made to other similar equipment to prevent this unforeseen loading from occurring in the future. Without the ECA to calculate the significance of the defect, the failure investigation might not have identified the real cause of the failure, risking other similar failures to occur.

Finally, an ECA can be used to justify a change of service or a life extension plan. Many structures are designed with an arbitrary 25 year design life. Once that time has elapsed it may be necessary to consider its continued service or change of service. Following a rigorous inspection of the structure, any defects present or flaws which may have grown over that time are identified and, if necessary, removed. The ECA can then quantify the further crack growth expected from additional cyclic service or determine the effect of increasing the stress in a structure in order to assess whether it is fit for further service.

11.4 Standards for assessment methods

The results of an ECA are often applied to safety critical components and it is, therefore, important to be sure that the assessment method is robust. For this reason, ECAs are carried out to a recognised standard and there are several procedures available.

A general procedure is given in BS 7910, 'Guide to methods for assessing the acceptability of flaws in metallic structures', which is applicable to all metallic materials, components and welds and gives extensive guidance on the assessment of fracture and fatigue. This procedure was developed and maintained by the British Standards Institution (BSI) in the UK. Harmonised software packages are available to assist in carrying out ECAs and an example is shown in Figure 11.1.

For pressure equipment fabricated in accordance with the ASME fabrication codes an alternative is the American code API 579-1/ASME FFS-1 'Fitness-for Service'. This code gives assessment methods for a wider range of in-service damage as well as fracture, and includes a basic level of assessment intended for inspectors on site to make quick decisions, as well as more specialist methods to be used by fracture mechanics experts.

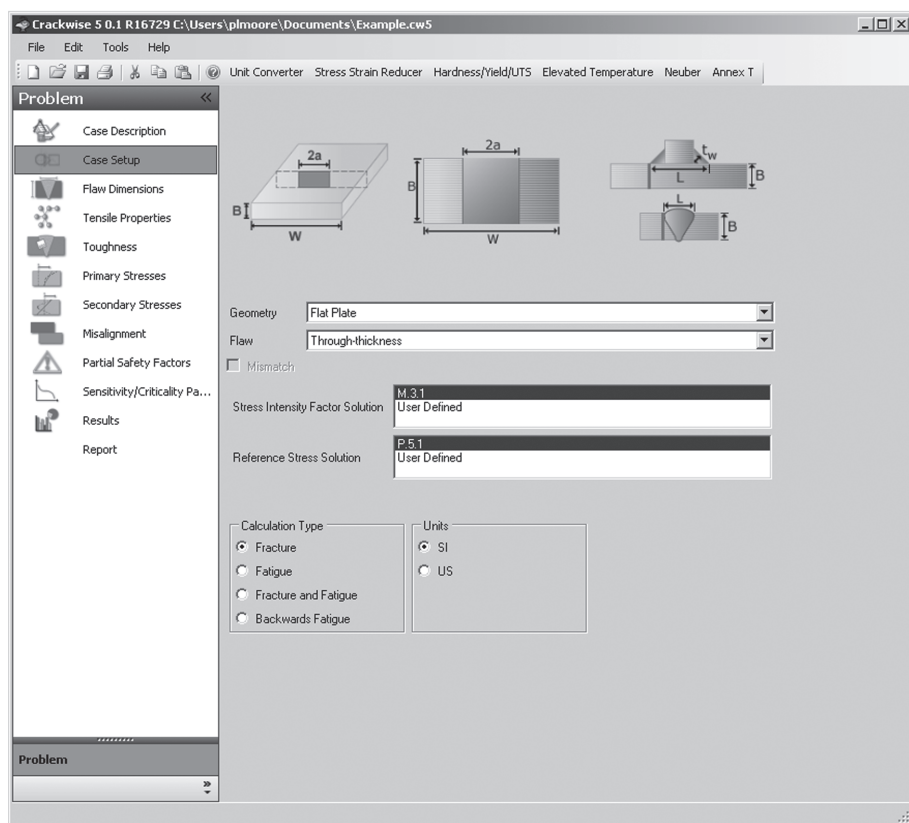
The British nuclear industry also has its own fitness-for-service assessment code developed for the fracture assessment of nuclear equipment called R6 'Assessment of the integrity of structures containing defects', initially developed by British Energy, and now maintained by a panel of industry experts.

These various assessment procedures differ in some significant ways, but the principles of performing an ECA are the same. These common principles are discussed in this chapter and Chapter 12, although most specific examples are taken from BS 7910. All of these ECA procedures require an expert in fracture assessment procedures to implement them accurately. For a welding engineer, it is necessary to have an

appreciation of the nature of the assessment methods, without needing to perform the assessment themselves. Those who may need to perform ECAs for themselves should seek out more information from the relevant fitness-for-service assessment procedure itself, and any associated software and training courses.

11.5 Input data for ECA

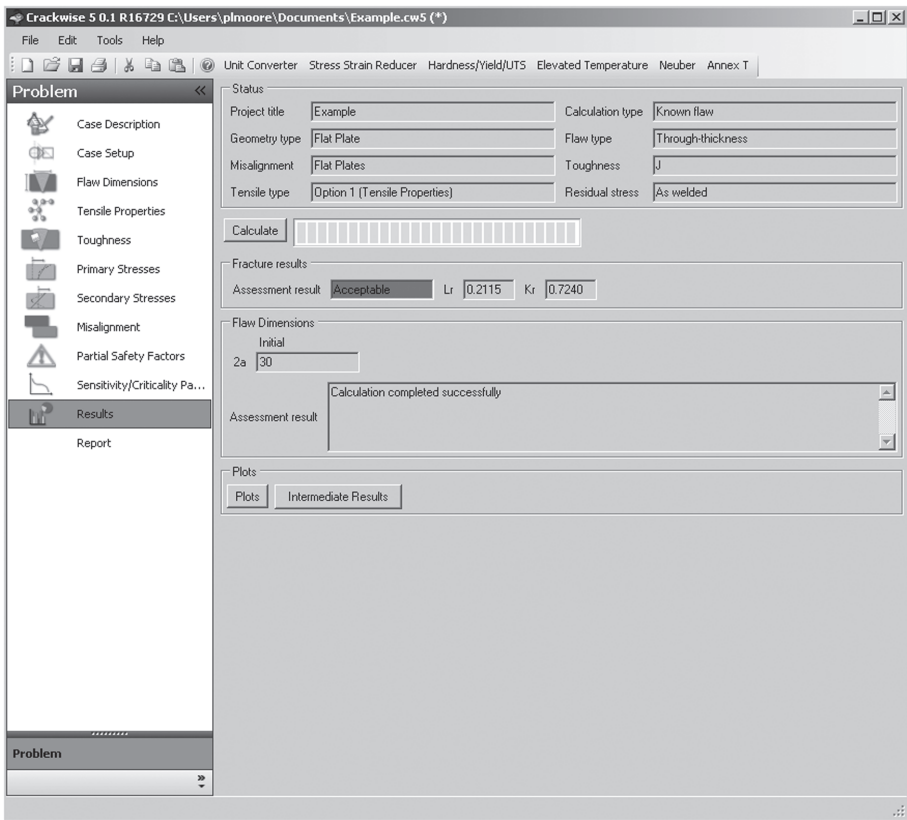
A significant amount of input data is required to perform an ECA, because separate calculations are made about the load-bearing capacity of the structure and the crack-driving force for the defect, as well as any fatigue-crack growth calculations. Each



(a)

Figure 11.1 Screen capture images of TWI's CrackWISE software: (a) the fracture case setup shows a through-thickness flaw in a flat plate (© TWI).

Continued



(b)

Figure 11.1 Continued. (b) the results show that a 30mm long flaw is acceptable (© TWI).

of these calculations is based on a number of different parameters. The better the quality of the input data (i.e. the fewer assumptions or conservative estimates used), the more accurate the results of the ECA are.

The set of input data can be broadly divided into three main groups: flaws and geometry, materials properties, and stresses. The input data required for an ECA can therefore be remembered in the form of a triangle (Figure 11.2), which is similar in some ways to the causes of brittle fracture discussed in 5.2.2

11.5.1 Flaws and geometry

The first stage of the ECA is to define the size and shape of the structure to be assessed. This could be a flat plate or a pipe or a solid bar of material. The thickness of the material must be known as well as the diameter if the component is cylindrical (such as for circumferential welds or girth welds). It is also necessary to define the

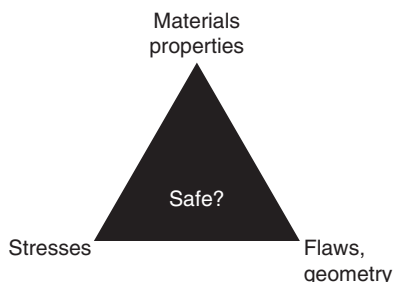


Figure 11.2 The three main sets of input data required for a fitness-for-service assessment or ECA.

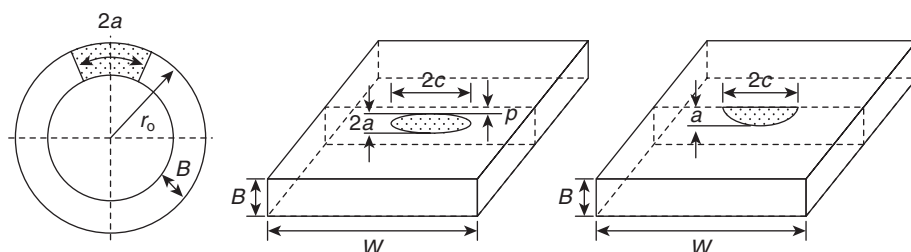


Figure 11.3 Typical flaw geometries assessed in an ECA: a through-thickness flaw in a cylinder, an embedded flaw in a plate and a surface breaking flaw in a plate, taken from BS7910:2013 (© TWI).

flaw type and its dimensions. Welding fabrication flaws are considered to be either planar or volumetric. In terms of risk of fracture and fatigue, it is the planar flaws that are of most concern. In an ECA, the flaws being assessed are assumed to be planar, behaving like cracks. In BS 7910 the flaw is categorised as either a through-thickness flaw, an embedded (buried) flaw, or a surface-breaking flaw (either on the weld cap or weld root). Depending on the type of flaw, it is necessary to define the flaw length, as well as the flaw height and the depth by which it is buried (called the ligament height). Real flaws might be irregular in their shape, but for the assessment, surface-breaking flaws are modelled as semielliptical in shape with a flaw height a and a surface length $2c$. Embedded flaws are assumed to be elliptical in shape with a height $2a$ and a length $2c$. Through-thickness flaws are assumed to have straight crack fronts, and are defined with a flaw length $2a$. Some typical flaw geometries that are assessed in an ECA are shown in Figure 11.3.

The ECA itself does not make allowance for any NDT sizing error in the flaw size; such errors depend on the inspection method used to detect the flaws. The flaw size used in the ECA may be an assumed size used as the basis for critical flaw assessments, the final values of which are reduced by an amount equal to the flaw sizing error

before implementation. Alternatively, for the assessment of a known flaw size, the reported flaw size must be increased by the flaw sizing error to reflect the size of flaw that could possibly be in the structure. More information on the detection of defects and NDT methods is given in Chapter 10.

For the assessment of welded joints in BS 7910, it may also be necessary to specify the width of the weld cap, in order to account for the stress magnification effect at the weld toe. Similarly, the maximum level of misalignment at the joint is required in order to determine the additional effective bending stress that this imposes on the joint.

11.5.2 Material properties

The material properties data required for an ECA are both the tensile properties and the fracture toughness of the material where the flaw is situated. The best results are given if full stress–strain curve data can be used in the assessment, rather than simply yield and ultimate tensile strength (UTS). If the stress–strain curve shows a yield discontinuity then that makes a difference to the assessment, particularly when the applied stresses approach yield. It is, therefore, important to know the stress–strain behaviour of the material.

The material toughness should be generated from fracture-toughness tests performed on representative weld and heat-affected zone (HAZ) notched specimens, tested at the appropriate temperature. If the material is ductile, an R-curve can be generated instead of simply a single value of fracture toughness, thus improving the ECA. If fracture-toughness test data is not available, it is possible to make a conservative estimate of the fracture toughness from Charpy data, but this results in a degree of overconservatism in the assessment results. Mechanical tests are discussed in greater detail in Chapter 9.

11.5.3 Stresses

In order to perform the ECA, it is necessary to know the maximum membrane stress across the flaw. Sometimes this is the maximum design stress, or it can be obtained from a stress analysis of the structure. There might also be bending stresses acting on the flaw. If the loading is not purely axial, the stress is separated into membrane and bending components for the ECA. If the component is operating under cyclic loading, then the stress spectrum in terms of a set of stress ranges, and the number of cycles at each stress range should be input to the assessment. Further details of fatigue assessment are given in Chapter 12.

The stress is magnified by any stress concentrations at the weld. The various ECA procedures account for additional stress concentration effects in slightly different ways. Misalignment in the weld joint causes an additional bending stress and the size of the weld cap forms stress concentration effects at the weld toes.

The stresses imposed externally on the joint are called primary stresses, and contribute to the susceptibility to both ductile overload and brittle fracture. There are further stresses that might act on the weld, called secondary stresses. Examples of

secondary stresses include weld residual stress and thermal stresses. Secondary stresses are internal stresses which self-equilibrate inside the structure; they will not cause the component to move or yield. However, secondary stresses are detrimental to the structural integrity of a weld because they contribute to the likelihood of brittle fracture. This means that a welded structure under no external loading at all, in its as-welded state (with residual stresses up to yield strength magnitude) can suffer spontaneous brittle fracture (like the Liberty Ship failures discussed in Chapter 3).

If the weld has been subjected to PWHT then the residual stresses are reduced. Although accurate quantification of residual stresses is challenging and as-welded residual stresses are assumed to be as high as yield in tension, after PWHT using optimum heat treatment parameters, the residual stresses can be assumed to have decreased to as low as 20–30% of the yield strength.

11.6 Failure assessment diagrams (FAD)

The ECA procedure presents on a graph the proximity of the assessment conditions to brittle and ductile failure. The graph is called a failure assessment diagram (FAD) (Figure 11.4). The FAD is essentially a presentation of a failure assessment line based on the tensile properties of the material being assessed. Each specific flaw is then assessed numerically in terms of ductile overload or brittle fracture, and a point is plotted on the FAD based on these values. If the assessment point falls inside the failure assessment line then these conditions are acceptable. If the point is outside the assessment line then the conditions cannot be shown to be safe and, thus, are potentially unsafe (Figure 11.4). The specific equation for plotting the failure assessment line is given in the ECA standard. If the material's full stress–strain curve data are available, it is possible to plot a material-specific FAD, which gives more accurate ECA results (Figure 11.5).

The proximity to brittle fracture is plotted on the Y -axis, as a parameter called the fracture ratio K_r . The fracture ratio is, in simple terms, the crack-driving force (expressed as a stress intensity factor, see Chapter 5) divided by the material's fracture toughness. When there are both primary and secondary stresses acting on the flaw, the fracture ratio also incorporates an additional plasticity correction factor to account for the interaction of these two components of the stress. Once the crack driving force is greater than the fracture toughness, the fracture ratio becomes greater than one and brittle fracture may occur; the failure assessment line, therefore, crosses the Y -axis at a value of one.

On the X -axis, the proximity to ductile failure (plastic collapse) is plotted using a parameter called the stress ratio L_r , which is the ratio of the stress in the remaining cross-section area of material around the defect (called the 'reference stress'), divided by the average of the yield and UTS. The material does not suffer final ductile rupture until the UTS has been exceeded. Therefore, depending on the tensile properties of the material being assessed, the failure assessment line cuts the X -axis at greater than one (depending on the ratio of the UTS to the yield strength).

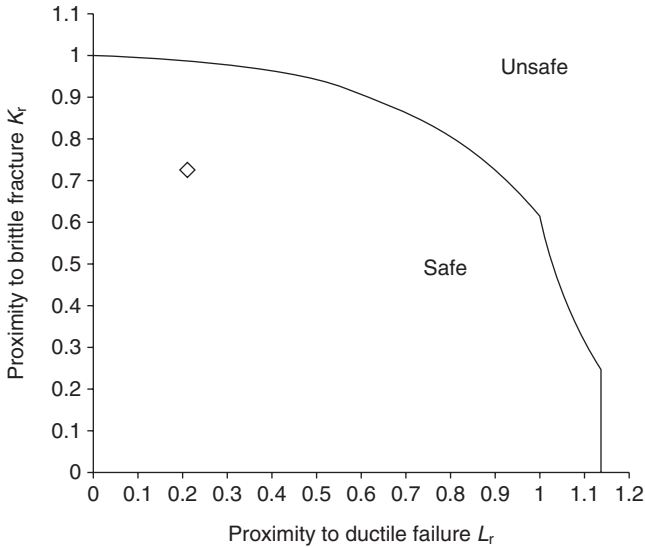


Figure 11.4 Assessment point for a 30 mm long through-thickness flaw in a 2 m wide and 50 mm thick Grade 450 plate with fracture toughness J of 156 kJ m^{-2} , shown on a failure assessment diagram (FAD).

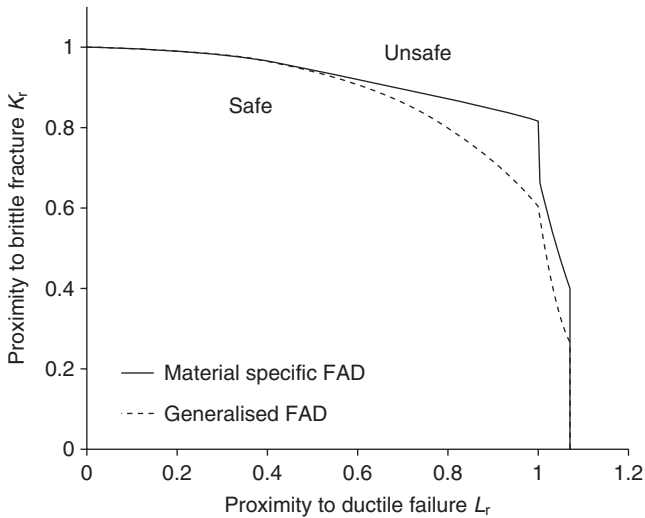


Figure 11.5 Generalised FAD and material-specific FAD from BS 7910 for a steel with a yield strength of 450 MPa and a UTS of 515 MPa, which shows continuous yielding on the stress–strain curve, rather than a yield plateau. The material-specific FAD is larger, meaning there is a larger safe region and less conservatism in the assessment.

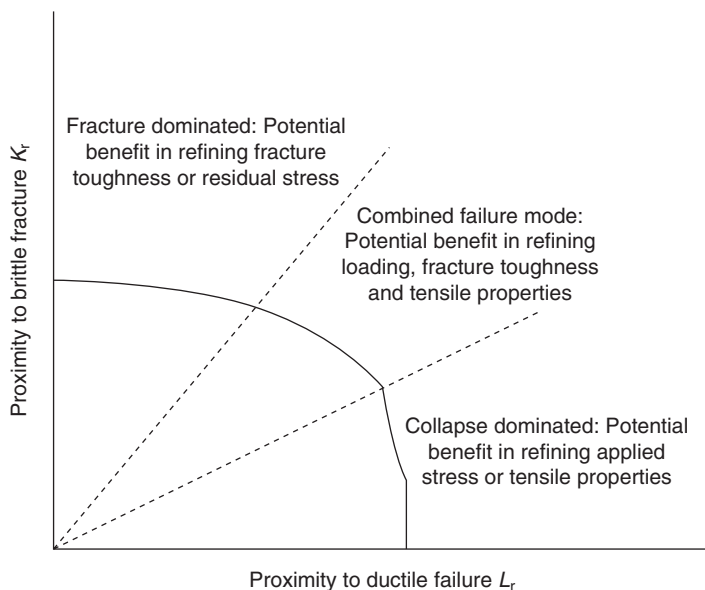


Figure 11.6 Regions of the FAD and the expected failure mode and most significant input parameters for assessment points lying close to or beyond the failure assessment line in those regions.

To determine the assessment point co-ordinates K_r and L_r for a particular case, it is necessary to be able to calculate the stress intensity factor and the reference stress, respectively. The formulae to calculate these are specific to a particular geometry of component and flaw type (such as a through-thickness flaw in a plate, or a surface flaw in a cylinder). However, for many common geometries such as these, the stress intensity factor solution and the equivalent reference stress solution are published in fracture mechanics handbooks or in fitness-for-service assessment procedures.

The location of the assessment point within the FAD can also provide information about the expected failure mode (Figure 11.6). An assessment point close to the Y-axis with a high value of K_r shows that the performance of the joint is fracture dominated, and that the low value of fracture toughness or possibly the high residual stresses are dominating the performance of the joint. An assessment point close to the X-axis of the FAD with a high value of L_r shows a plastic-collapse-dominated assessment, where the structure has high toughness, but experiences stresses close to or beyond the yield strength of the material, and the tensile properties limit defect tolerance. An assessment point close the failure assessment line in the 'knee' region corresponds to a potentially mixed mode failure, where both the fracture toughness and tensile properties of the material affect the risk of failure. An assessment point which has low values of K_r and L_r is under very low loading, and is not expected to fail by either fracture or collapse.

11.7 Proximity to failure and safety factors

Sometimes the ECA is carried out specifically in order to determine the critical value of one of the input parameters. The example shown in Figure 11.4 is for a 30 mm long through-thickness flaw in a 2 m wide and 50 mm thick Grade 450 plate with a yield strength of 450 MPa and a fracture toughness J of 156 kJ m^{-2} . A 30 mm long through-thickness flaw is tolerable in this case, but what is the longest through-thickness flaw that is still tolerable? Although the calculation can be performed manually, software packages are available to avoid tedious repetition, and it is straightforward to iterate the assessments to find out the maximum tolerable length of a through-thickness flaw, which in this case is 60 mm (Figure 11.7). The maximum tolerable flaw size is the critical assessment condition that just reaches the failure assessment line and the flaw is, therefore, shown to be safe. Alternatively, the critical parameter assessment may be used for a case where a given defect size is shown to be unsafe, outside the FAD, and it is possible to calculate by how much the stress on the component, for example, must be reduced so that the flaw is fit for service. Effectively, the calculation iteratively reduces the stress until the assessment point just touches the failure assessment line. The critical value of any input parameter can be calculated in this way.

There is always some degree of conservatism inherent in the procedures, because they are intended to guarantee that a structure is safe, rather than to predict failure. Simply because an assessment point lies outside the FAD does not necessarily mean

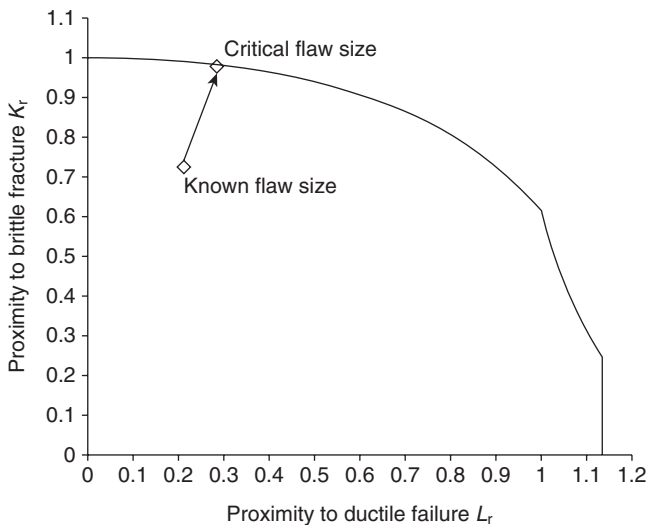


Figure 11.7 Critical through-thickness flaw assessment of a 2 m wide and 50 mm thick Grade 450 plate with fracture toughness J of 156 kJ m^{-2} . The initial known flaw 30 mm long is safe up to a critical length of 60 mm.

that it will imminently fail, it simply means that the assessment procedure cannot guarantee it is safe. Most ECA procedures require that conservative assumptions about input data are used in order to ensure safe assessments.

For an assessment point that lies inside the line, the closer the assessment point lies to the line, the smaller the safety factor on the assessment. The safety factor is calculated from the ratio of the critical conditions to the assessment conditions (i.e. the safety factor on flaw size is the critical flaw size divided by the actual flaw size and the safety factor on membrane stress is the critical stress divided by the actual membrane stress). In our example of the through-thickness flaw in a plate, the critical flaw length is 60 mm, whereas the actual flaw length is 30 mm, which therefore has a safety factor of 2 in the structure under these conditions.

11.8 Refining the assessment

If the assessment point from an ECA lies outside the FAD when a simple calculation is carried out then that is not always the end of the story in terms of rejecting that flaw. It is possible to refine the assessment in a number of ways in order to improve the accuracy of the assessment, potentially removing some over-conservatism and demonstrating that the flaw may be fit for service.

11.8.1 Accuracy of input data

The first option for refining the assessment is in the input materials data. If the tensile properties are supplied as simply yield and UTS values, then using the full stress-strain curve data to allow a material-specific FAD to be plotted gives some benefit. If the fracture toughness is a single value, but the material is ductile, then using a *J-R* curve instead allows the assessment to get some benefit from the stable tearing that occurs in ductile materials. An alternative approach is to use fracture mechanics specimens that have lower constraint than the standard deeply-notched single-edge notched bend (SENB) specimens, such as single-edge notched tension (SENT) specimens, provided the constraint is still lower than that of the flaw in the structure. Lower constraint specimens give higher values of fracture toughness when all other conditions are the same. This approach using SENT specimens is common for assessment of flaws in girth welds during installation. With regards to the fracture toughness, simply confirming whether the assumption about the minimum temperature is appropriate can be beneficial in terms of being able to use fracture toughness data tested at slightly warmer temperatures, which would be expected to give improved fracture toughness.

Second, the assumptions about the stresses can be reviewed. If the original assessment assumed the structure would experience the maximum allowable stress, this might be over-conservative if under normal operating conditions of the component the maximum operating stress is significantly below this. It might be possible to use strain gauges on similar equipment to measure more realistic stresses, particularly if the ECA is to determine flaw tolerance in regions that might not experience the maximum stresses in the whole component.

Similarly, there may be some benefit in revising the assumption about the secondary stresses, particularly for as-welded structures where the default assumption is yield magnitude tensile residual stress. BS 7910 provides examples of residual stress distributions for various weld types and heat inputs, which might be applicable to the assessment. If so, then if the flaw is located in a part of the weld where the residual stress is much less than yield, there is a significant benefit in the assessment.

Most of these measures require more effort in terms of seeking out more data, making justifications or spending more time to perform the assessment. It is good practice, however, to start the ECA using basic assumptions and then revise them where necessary, rather than starting off with the most complex assessment conditions, which might not be necessary.

11.8.2 Sensitivity studies

When an assessment point falls outside the FAD at the first assessment, and a refinement to the assessment is needed, it can be useful to carry out some sensitivity studies to identify which input parameters are the most critical. This is simple to do when using ECA software, which can graphically plot the effect of a change in one parameter on the tolerable flaw size. For example, a sensitivity study to show the effect of fracture toughness on tolerable length gives an indication of whether revising the assumptions about fracture toughness will give sufficient improvement in the permitted flaw size. If not, it could be that the limiting flaw size is more dominated by the fatigue crack growth or the proximity to plastic collapse, in which case other input assumptions are more sensitive and seeking out more accurate fracture toughness data may not significantly improve the tolerable flaw size. Figure 11.8 shows how the maximum tolerable flaw length in the 2 m wide 50 mm thick flat plate from Figure 11.4 alters with fracture toughness. The default condition gave a critical flaw length

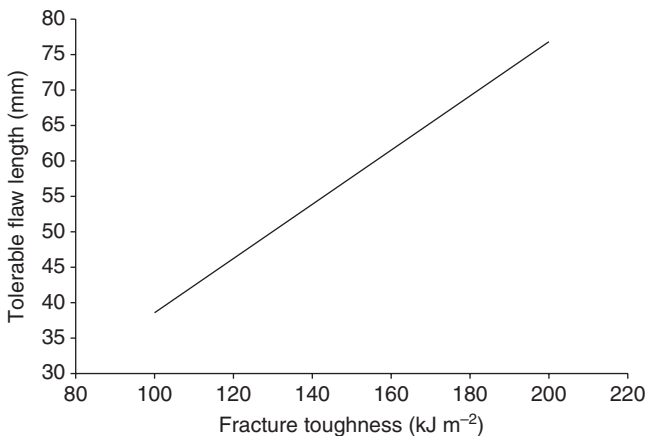


Figure 11.8 Sensitivity curve showing the effect of fracture toughness on the critical flaw length in the flat plate assessment shown in Figures 11.4 and 11.7.

of 60 mm with the fracture toughness of 156 kJ m^{-2} . If the toughness were only 100 kJ m^{-2} , the tolerable flaw length would be 38 mm, whereas an increase in toughness to 200 kJ m^{-2} increases the critical flaw length to 77 mm.

It is often beneficial to perform a few sensitivity studies for any ECA in order to assess the impact of small variations in the input data. However, the real power of performing sensitivity assessments is that they show the ECA engineer where to target refinements to the assessment.

11.8.3 *Selecting the assessment cases*

Sometimes an assessment uses the most conservative assumptions for the whole structure, and assesses one single case assuming that all those conditions coincide. It is more thorough if separate assessments are made for different regions, one of which may be under higher stresses or with thinner wall thickness, compared with other regions which may be under lower stresses or have greater wall thickness. Neither of these cases might be as conservative as assuming the worst case from each together in one assessment. Even if the worst cases do coincide in one location, separate ECA cases allow the more stringent flaw tolerance to be targeted at these most critical welds, whereas the rest of the fabrication might be able to tolerate larger flaws because they can be assessed by an ECA case which does not include the most severe conditions.

11.8.4 *Complexity of the assessment*

The final option to refine the assessment is to increase the complexity of the assessment method itself. Some fitness-for-service standards permit the direct use of numerical modelling for the assessment. A finite element model of the cracked geometry is generated and loaded to the appropriate level, both under primary and secondary loads. From the model, the crack-driving force on a crack of a given size is calculated, and this is then compared directly with the fracture toughness of the component that resists that crack-driving force. If the fracture toughness exceeds the crack-driving force, the component is acceptable, but if there is insufficient resistance from the material fracture toughness, compared with the driving force on the crack, then the conditions are not acceptable.

The amount of effort to make just one assessment case following this numerical modelling method adds extra cost compared with the main ECA methods given in the standards. However, it offers the greatest accuracy in the results. Using numerical modelling can also allow the assessment of more complex geometries where standard solutions for K_r and L_r may not exist.

11.9 Conclusions

An ECA for a given defect under a set of loading condition is carried out to assess the fitness-for-service of the weld in question containing that flaw. The flaw is

considered to be acceptable provided that the conditions for failure are not reached within the required service life.

Performing a fitness-for-service assessment for a flaw in a welded structure is not a simple calculation. It requires a great deal of knowledge and practice to know how to perform assessments based on the standard ECA procedures. There are degrees of complexity in the ECA and a basic initial assessment might offer scope for refinement with some revision of any initial conservative assumptions.

However, the ECA approach is enormously flexible, both in terms of when during the lifetime of a component it can be applied, and what it can demonstrate in terms of flaw tolerance and calculating the critical values of a range of input parameters. The accuracy of the ECA is highly dependent on the quality of the input data used and it is important that the welding engineer understands which component details, materials properties, and stress data have to be supplied in order to get the best results from the fitness-for-service assessment.

11.10 Sources of further information and advice

- Antaki, G (2005) *Fitness-for-service evaluations for piping and pressure vessels: ASME code simplified*, McGraw-Hill Professional, ISBN-10: 0071453997, ISBN-13: 978-0071453998.
- API 579-1 / ASME FFS-1 (2007) *Fitness-for-service, 2nd edition*, American Petroleum Institute & American Society of Mechanical Engineers.
- BS 7910 (2013) *Guide to methods for assessing the acceptability of flaws in metallic structures*, British Standards Institution.
- R6 (2001) *Assessment of the integrity of structures containing defects, Revision 4*, EDF Energy.

12.1 Introduction

In Chapter 7, we described how S–N curve data obtained experimentally are used to design welded joints against fatigue failure, and to assess the significance of the stress cycles experienced in service. The S–N curve approach is now well established as the conventional method for fatigue design. However, the S–N curve assessment method is used to estimate a design fatigue life for a weld meeting workmanship quality levels to ensure avoidance of failure. The S–N curve approach, has a major limitation because once a flaw or crack has been detected it provides no information about the growing fatigue crack. This is quite a serious omission as we know that fatigue failure occurs by the steady progressive growth of a crack across the load bearing member. To assess fatigue crack growth, fracture mechanics principles, developed in the 1950s and 1960s, are used. This can also be combined with a fitness-for-service assessment, as described in the previous chapter.

12.2 Using fracture mechanics to describe fatigue crack growth

There are two aspects to the use of fracture mechanics to describe fatigue behaviour. First, we consider what we mean by fatigue crack growth and, second, we find a way of linking that crack growth to a specific crack experiencing a particular fatigue loading cycle.

12.2.1 *Fatigue crack growth*

Metallurgical studies of fatigue fracture surfaces in alloys such as aluminium have shown the presence of closely spaced marks called striations. These striations are very regular and are generally considered to correspond to the position of the fatigue crack front as it progressively grows through the material. This leads to the concept that the fatigue crack extends by a very small amount in each fatigue cycle, Figure 12.1.

If we assume that in one cycle, the fatigue crack extends by a small amount Δa , then its new length becomes $a + \Delta a$. The extension per cycle is known as the fatigue

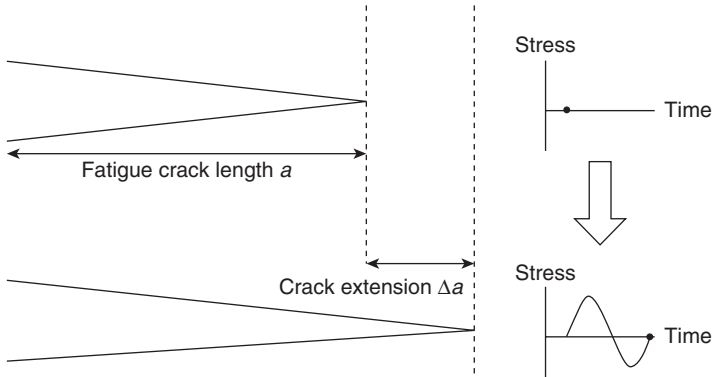


Figure 12.1 Fatigue crack extension over one stress cycle.

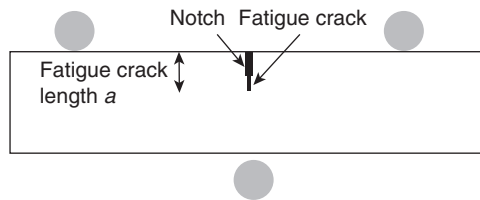


Figure 12.2 Simple fatigue crack growth rate test, showing a notched specimen loaded in three-point bending under a given constant amplitude cyclic stress range.

crack growth rate. It is important to note that the use of the word ‘rate’ does not mean per second (or any other unit of time); it refers to each fatigue cycle, no matter how fast or slow the frequency of loading.

N denotes the number of cycles, and the fatigue crack propagation rate per cycle is expressed as da/dN in mm/cycle.

Fatigue crack growth rate in a material can be measured from tests on a range of specimen geometries, as described in Section 9.7. Figure 12.2 shows a fatigue crack growing in a specimen loaded in three-point bending.

Under cyclic loading, the fatigue crack steadily grows across the specimen. At any time, the fatigue crack length can be measured using, for example, a travelling microscope. Other more accurate techniques measure the change in electrical resistance as the crack grows (and the area through which the current passes decreases). In this way, a graph of fatigue crack length against number of cycles can be obtained, Figure 12.3. At any particular value of crack length, say a_1 , a tangent can be drawn on the graph. This tangent represents the slope of the curve and, hence, is a measure of da/dN , the fatigue crack growth rate. In almost all circumstances, the rate of fatigue crack growth increases as the crack length increases, as can be seen by drawing a second tangent at a crack length of a_2 in Figure 12.3.

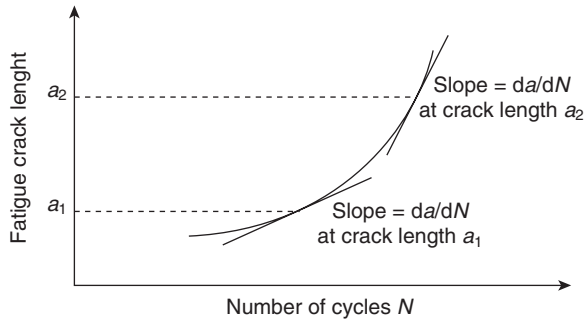


Figure 12.3 Fatigue crack length increasing with the number of cycles. The slope at any point gives the crack growth rate.

12.2.2 Stress intensity factor

In Chapter 5, we introduced the stress intensity factor K associated with a crack length a in a body subjected to a stress σ . The stress intensity factor was defined as:

$$K = Y\sigma\sqrt{\pi a} \quad (12.1)$$

where Y is the geometry term depending on the type of loading, and details of the crack and of the stressed member. The original mathematics made the simplifying assumption that the crack was in an infinitely large plate subjected only to axial stress. The term Y simply takes into account the more realistic practical case concerning modestly sized geometries, not infinitely large plates. The stress intensity factor K completely defines the detailed stress distribution in the vicinity of a crack. Thus, for two apparently different cases: a long crack subjected to a small applied stress and a short crack experiencing a high applied stress in a component of a different geometry, if the stress intensity calculated from equation 12.1 is the same, then the stress distributions near the crack are the same. The behaviour of the crack, in particular whether it propagates or not, can reasonably be expected to be governed by the stress distribution near the crack, which is itself only dependent on the stress intensity factor K .

The stress intensity factor approach was originally developed for a crack under a constant stress, and can be used as a method of calculating the likelihood of brittle fracture (see Chapters 5 and 11). The stress intensity factor is also useful in assessing fatigue behaviour. Under cyclic loading the applied stress σ varies with time and, thus, the stress intensity factor also varies with time, as shown in Figure 12.4.

We can identify a maximum stress, minimum stress and stress range. Thus, using the simple expression in equation 12.1, we can similarly identify the maximum stress intensity factor in the fatigue cycle, the minimum stress intensity factor, and the stress intensity factor range.

$$K_{\max} = Y\sigma_{\max}\sqrt{\pi a} \quad (12.2)$$

$$K_{\min} = Y\sigma_{\min}\sqrt{\pi a} \quad (12.3)$$

$$K_{\max} - K_{\min} = \Delta K = Y\Delta\sigma\sqrt{\pi a} \quad (12.4)$$

An additional parameter, the stress ratio R , is defined as

$$R = \frac{K_{\min}}{K_{\max}} = \frac{\sigma_{\min}}{\sigma_{\max}} \quad (12.5)$$

Stress ratio is often used as a way of assessing the effect of mean stress; for the same value of ΔK an increasing value of R corresponds to an increasing value of mean stress.

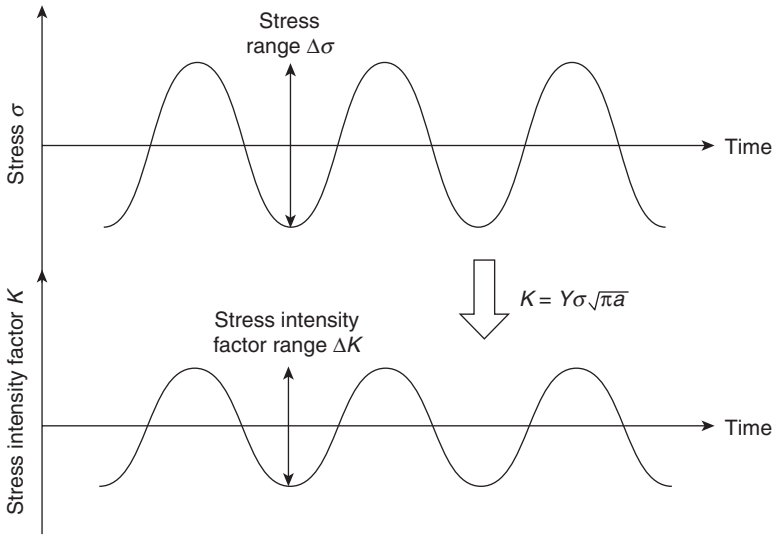


Figure 12.4 Variation of stress and stress intensity factor with time.

12.3 The power law (Paris equation)

We are now in a position to use the two parameters described in 12.2, the rate of crack propagation per cycle (da/dN) and stress intensity factor range (ΔK). Many experimental measurements from the 1950s onwards have shown that, for a fatigue crack growing steadily in a material, the relationship between da/dN and ΔK is as shown schematically in Figure 12.5, plotted with logarithmic axes.

Over a wide range of ΔK , there is a linear relationship between $\log da/dN$ and $\log \Delta K$. Using the conventional definition of a straight line, $y = mx + c$, this region of fatigue crack growth behaviour can be written as:

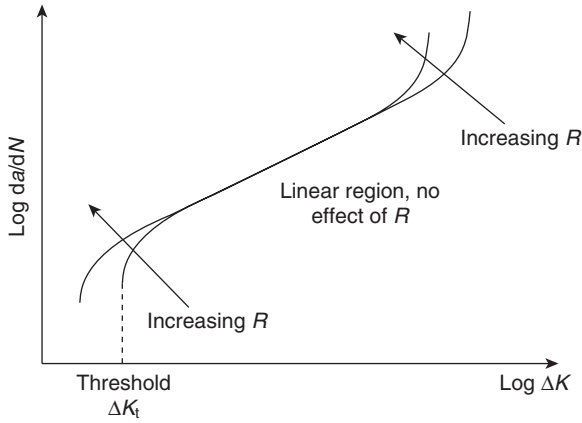


Figure 12.5 Relationship between da/dN and ΔK plotted on logarithmic scale axes.

$$\log \frac{da}{dN} = m \log (\Delta K) + \log A \quad (12.6)$$

or

$$\log \frac{da}{dN} = \log (\Delta K)^m + \log A \quad (12.7)$$

These terms can be combined to give:

$$\log \frac{da}{dN} = \log [A (\Delta K)^m] \quad (12.8)$$

Then removing the logarithm from both sides gives:

$$\frac{da}{dN} = A (\Delta K)^m \quad (12.9)$$

where A and m are constants.

Equation 12.9 is the fundamental basis of the fracture mechanics approach to describing fatigue crack propagation. It is also known as the 'power law' or the Paris equation, named after the engineer who proposed it.

The constants A and m are material dependent and have been measured experimentally for many materials. Thus, if the crack length a and stress range are both known, ΔK can be calculated from equation 12.4 and, thus, the rate of crack propagation per cycle can be calculated.

In the region where the power law applies, there is no influence of the stress ratio (i.e. mean stress), and the rate of crack propagation depends entirely on the stress intensity factor range. At high values of ΔK , the material behaviour deviates from the power law and the rate of fatigue crack growth is faster than that given by the power law. This is because the maximum stress intensity factor ($K_{\max}$) starts to approach the material fracture toughness K_{mat} . The fatigue crack no longer propagates entirely because of a fatigue mechanism; other modes of crack extension that are associated with static fracture, such as ductile tearing, also contribute to the crack growth. For the same stress intensity factor range, a higher value of mean stress (or stress ratio) gives rise to a higher value of $K_{\max}$ and, hence, a greater incidence of additional modes of crack extension. At high values of ΔK , therefore, crack propagation rate depends on the mean stress R .

At very low values of ΔK , the rate of crack propagation becomes lower than would be expected by a continuation of the power law. Indeed, the rate of crack propagation becomes lower and lower, and there is a critical value of stress intensity factor below which fatigue crack growth does not occur. This is known as the fatigue threshold and the critical value of ΔK is expressed as ΔK_I . The fatigue threshold depends on mean stress, or stress ratio R , with an increasing mean stress causing the threshold to fall to a lower value.

Determining fatigue crack growth rate curves for a particular material is relatively simple. A specimen of the material, such as that shown in Figure 12.2 is used to obtain a graph of crack length against number of cycles, as shown in Figure 12.3. When the crack length is a , the rate of crack extension can be calculated, also as shown in Figure 12.3, as da/dN at crack length a_1 . The next step is to determine the stress intensity factor range ΔK corresponding to the crack length a_1 using:

$$\Delta K = Y\Delta\sigma\sqrt{\pi a_1} \quad (12.10)$$

For standard geometries, Y , $\Delta\sigma$ and a_1 are known and, hence, ΔK can be calculated. Thus, once we have determined ΔK , the corresponding da/dN can be calculated, i.e. one data point to plot on the crack propagation curve. We can follow exactly the same procedure to calculate da/dN and the corresponding values of ΔK at different values of crack length, such as a_2 . In this way, a great deal of crack propagation data can be obtained from a single specimen.

12.3.1 Power law constants

As is the case with S–N curve data, measurements of the fatigue crack growth rate show a degree of statistical scatter. Typical experimental results for the linear region are shown in Figure 12.6. Regression analysis is used to draw the mean line corresponding to the results and, when the fatigue crack growth characteristics of different grades of steel are measured, there is no difference in growth rates. Furthermore, fatigue cracks grow at the same rate in parent steel, in weld metal, and in all parts of the heat affected zone (HAZ). The rate of fatigue crack propagation is totally independent of steel microstructure.

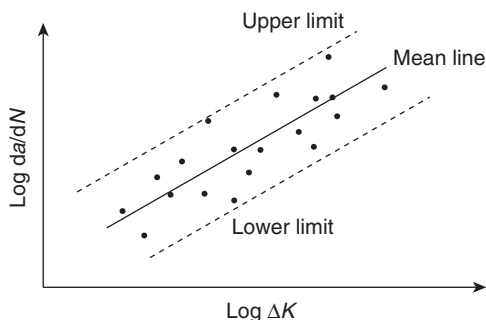


Figure 12.6 Schematic diagram illustrating the statistical scatter in fatigue crack growth rate measurements.

For design purposes, the mean value of the crack propagation rate is not appropriate; instead, we use the upper limit of the data, i.e. we assume the fatigue crack grows at its fastest possible rate. This approach is analogous to the use of the lower limit of S–N data rather than the mean. There are several ways of estimating the upper limit of fatigue crack growth rate data and commonly accepted values for steel are $m = 3$ and $A = 5.21 \times 10^{-13}$.

These values can be substituted into equation 12.9, where da/dN is in mm/cycle and ΔK is in $\text{Nmm}^{-3/2}$. If other units for ΔK and da/dN are chosen, such as $\text{MPa}\sqrt{\text{m}}$ and m/cycle, the value of the constant m remains 3, but the value of A depends on the new units and m and must be calculated.

An extensive body of fatigue data has also been developed for many aluminium alloys. Because damage-tolerant design is used extensively to ensure the integrity of aluminium fuselages and other structures by the aircraft industry, fatigue data for aluminium alloys is generally available in the open literature. Therefore, it is often possible to find appropriate test data when values of A , m and ΔK , are required for aluminium. The slope m of the linear region of the crack growth curve, however, is usually approximately 3.

12.3.2 Applying the power law

If we consider a fatigue crack of length a growing under constant amplitude loading with a stress range $\Delta\sigma$, we can calculate ΔK using equation 12.4. The rate of crack extension per cycle can then be calculated using equation 12.9. If only one cycle is considered, we can calculate the very small amount by which the crack grows. For the second cycle, the crack is now slightly longer, and we can calculate a new, slightly greater value of ΔK . This enables us to calculate the slightly greater, but still very small amount by which the crack has extended. This new crack length can again be used to calculate a new ΔK and, thus, the amount that the crack extends in a third cycle. By continuing this approach for cycle after cycle, it is possible to calculate by

how much a fatigue crack extends after a given number of cycles. The manual cycle-by-cycle procedure described above is very time consuming and tedious; the procedure, however, is equivalent to mathematical integration:

$$\frac{da}{dN} = A(\Delta K)^m \quad (12.11)$$

Substituting for ΔK using the expression in equation 12.10

$$\frac{da}{dN} = A[Y\Delta\sigma\sqrt{\pi a}]^m \quad (12.12)$$

and then rearranging gives

$$\frac{1}{[Y\sqrt{a}]^m} \frac{da}{dN} = A[\Delta\sigma\sqrt{\pi}]^m \quad (12.13)$$

Equation 12.13 can now be integrated, bearing in mind that $a=a_1$ when $N=0$ cycles (at the beginning of fatigue cycling), and $a=a_2$ when $N=N$ cycles (at the end of the fatigue cycling).

$$\frac{1}{A[\Delta\sigma\sqrt{\pi}]^m} \int_{a=a_1}^{a=a_2} \frac{da}{[Y\sqrt{a}]^m} = \int_{N=0}^{N=N} dN = N \quad (12.14)$$

Equation 12.14 may look a little daunting, but there are computer programs available to evaluate the integral and, therefore, the equation becomes arithmetic. Equation 12.14 is the backbone of the use of fracture mechanics to calculate fatigue crack growth. In this equation, the values of A , π , and m are known, and Y depends only on crack length for most cases and is also known. There are therefore four variables: stress range $\Delta\sigma$, initial crack size a_1 , final crack size a_2 and the number of cycles to grow from a_1 to a_2 , N .

If any three of these parameters are known, then equation 12.14 can be used to calculate the fourth. For example, if we know how many stress cycles in a given stress range are applied to a crack of a known size, we can calculate the upper limit to the crack size after fatigue cycling. We can, therefore, compare this upper limit of the crack size with that determined acceptable by other criteria such as brittle fracture or plastic collapse (see Chapter 11). As a second example, we may know the initial crack size, the final allowable flaw size and the stress range. In this case, we can calculate the number of cycles associated with the crack growing from its initial size to its final size. Third, if we know the final allowable flaw size, the stress range, and the applied number of cycles, we can calculate the maximum allowable size of any initial flaw. The welding procedures and inspection regime must then be used to ensure that no flaws greater than this size exist in the fabrication.

12.4 Assessing weld flaws under fatigue loading

The principles described in Section 12.3 can be used to assess the growth of a fatigue crack, should one be found in service. The same principles can also be used to assess planar weld flaws such as hydrogen cracks and lack of fusion flaws. The assumption is made that the weld flaw behaves as though it were a fatigue crack. This is justified because microscopic examination of different types of flaw and crack show that the tip of a fatigue crack is sharper than most weld flaws; only the tip of a hydrogen crack is as sharp as a fatigue crack. In reality, the assumption that all planar flaws are treated as though they were fatigue cracks therefore builds in an additional level of conservatism.

Equation 12.15 is used, where a_1 is the initial flaw length and a_2 is the maximum tolerable flaw length based on, for example, the risk of brittle fracture or plastic collapse from an ECA (Chapter 11). The applied stress range is $\Delta\sigma$ and N is the number of cycles associated with growing the crack from length a_1 to a_2 .

$$N = \frac{1}{A[\Delta\sigma\sqrt{\pi}]^m} \int_{a_1}^{a_2} \frac{da}{[Y\sqrt{a}]^m} \quad (12.15)$$

A common situation that involves possible extension of a flaw by fatigue is when a planar weld flaw is found, either late in fabrication or during service. The power law and equation 12.15 can be used to establish the lifetime, or allowable number of cycles, before further action such as repair is needed. In some cases, the initial value of ΔK is so small that it is below the fatigue threshold ΔK_t . In these circumstances, the flaw does not extend by fatigue and no action is required.

12.5 Advanced fatigue crack growth assessment

Fatigue cracks are assessed under various environmental conditions and different loads.

12.5.1 Environmental effects

The fatigue crack growth rate in steel is independent of the steel microstructure, but other factors such as the presence of a corrosive environment (e.g. seawater) or operation at temperatures above 100°C influence the crack growth rate. Fatigue cracks in welds exposed to seawater and marine environments propagate faster than in welds simply exposed to air. Values of the constants A and m in equation 12.9 are published for a wide range of different materials (including steels, aluminium alloys and titanium alloys) and a wide range of environments. If power law constants for the required combination of material and environment are not available then fatigue crack growth rate testing can be carried out using specimens immersed in the environment of concern (see Section 9.7).

12.5.2 Variable amplitude loading

The power law expression refers to cracks growing under constant amplitude loading. Fatigue crack growth under variable amplitude loading can be assessed by using a stress cycle counting technique (such as the reservoir method or the rainflow method) to break the stress history down into blocks of constant amplitude loading (as described in Section 7.5). Each of these blocks is then considered separately and in turn to estimate the total crack extension by fatigue. It is assumed that the sequence in which the constant amplitude blocks are applied does not influence crack growth. In essence, this approach is directly equivalent to Miner's law of linear accumulative damage. There are, however, results that show that the sequence of loading (whether going from low stress ranges up to high, or starting with the high stress ranges going to lower) can have an effect and, therefore, care must be taken when assessing variable amplitude loading.

12.6 Conclusions

When cracks are detected and are known to be subject to fatigue loading, it is important to determine how quickly the crack could grow. This is a vital part of the fitness-for-service assessment of structures under fatigue loading. Extension of a crack by fatigue can be determined using fracture mechanics principles and the Paris power law. The rate of fatigue crack growth is the same for all grades of steel but depends on the environment the crack is growing in.

12.7 Sources of further information and advice

- BS 7910 (2013) *Guide to methods for assessing the acceptability of flaws in metallic structures*, British Standards Institution.
- Gurney, T. (2006) *Cumulative damage of welded joints*, CRC Press, ISBN-10: 0849334756, ISBN-13: 978-0849334757.
- Milella, P. P. (2012) *Fatigue and corrosion in metals*, Springer, ISBN-10: 8847023351, ISBN-13: 978-8847023352.
- Sofronas, A. (2012) *Case histories in vibration analysis and metal fatigue for the practicing engineer*, Wiley-Blackwell, ISBN-10: 1118169468, ISBN-13: 978-1118169469.

Improving the fracture performance and fatigue life of welded joints

13

13.1 Introduction

When assessing the engineering life of a welded structure, all potential failure modes must be considered and appropriate load limitations or other restrictions imposed. In this book, we have considered design for static loading, based on strength and toughness, and design against fatigue loading. There are a number of principles that should be considered in order to reduce the risk of ductile or brittle fracture. However, in many cases, it is design against fatigue that is the limiting case; in other words the static and fracture design criteria are automatically satisfied by the fatigue design limitations.

Fatigue strength may be improved by a number of technologies that are applied after the joint has been welded, and these are described later in this chapter. There are, however, several measures to improve fatigue performance that can be taken before the joint is welded. If, at the design stage, a fatigue analysis demonstrates that a particular joint does not possess adequate fatigue performance, then the following actions may be very cost effective and should not be overlooked. Similarly, there are some principles which can be followed at design or after welding to reduce the risk of fracture.

13.2 Fatigue improvement measures to be taken before welding

A number of measures can be taken before welding in order to improve the fatigue strength.

13.2.1 *Restricting the service loading*

By restricting the severity of service loading, the variable amplitude stress history is reduced. The majority of the design S–N curves have a slope of $-1/3$ (on logarithmic axes) so that a reduction in stress range of 10% results in an increase in design life of more than 30%. A natural consequence of restricting the applied loads is that the function and usefulness of the structure may be impaired.

13.2.2 Limiting the service life

Limiting the service life is a fairly obvious option that also compromises the usefulness and economic benefits of the structure. This measure, and the previous one, are included for completeness; neither is realistic in the majority of cases.

13.2.3 Reducing the applied stress range

The stress range is reduced by increasing the section thickness, thus reducing the stress range when either the same axial load or bending moment is applied. Increasing material thickness, however, increases the weight of the structure, which may in turn increase the applied load to be carried by the joint. It also increases the cost of the material required and makes fabrication costs higher.

Note that for the same stress range, fatigue life is reduced as material thickness increases, because, although the crack has further to grow, it grows at a faster initial rate. It is necessary to consider this 'thickness effect' only when the thickness exceeds 25 mm for plate and exceeds 16 mm for tubular nodal joints. The applied stress range may also be reduced by changing the structure's design such that the joint is located in an area of lower stress or further away from a stress concentration.

13.2.4 Using a joint with a higher fatigue classification

We have seen in Chapter 7 that the design S–N curve appropriate for a particular joint (i.e. the fatigue classification) depends on the detailed geometry. This is particularly important in the case of a transverse butt weld where joint class depends on whether the joint was made from one side (lower classification) or two sides (higher classification). If the joint is made from one side, the class depends on whether it was made using a backing strip or not, because backing reduces the fatigue classification. In a butt weld, the degree to which overfill is controlled reduces the fatigue class if it results in a poor weld profile.

Similarly, the geometry of an attachment on the surface of a stressed plate plays an important role in determining fatigue life. In particular, attachments near or at the end of the plate edge have poor fatigue classifications and should be moved to more than 10 mm from the edge. Likewise, the size of the attachment should be reduced whenever possible, limiting the length to less than 150 mm in the direction of the applied stress.

13.2.5 Avoiding welding

Avoiding welding may seem a strange recommendation to make in a book for welding engineers, but in some demanding fatigue applications, complex welded fabrications may simply not possess sufficient fatigue strength. Replacement of some or all of a welded fabrication by a casting (for instance) may mean that only butt welds are required and that these welds are in areas of relatively low stress. The casting can be designed to have varying wall thickness and smooth transitions and, thus, much better

fatigue performance than a welded fabrication performing the same function. Examples where steel castings or forgings have been used include excavator booms on earthmoving equipment and for tubular connections on offshore platforms.

The use of castings to reduce the number of welds is particularly attractive for fatigue designs in aluminium structures, which have relatively poor fatigue resistance but are easy to extrude, bend and machine into complex sub-assemblies.

13.3 Fatigue improvement techniques for welds

Most of the effort to develop post-weld fatigue improvement techniques has quite naturally concentrated on those joints that have the poorest fatigue performance. These joints are attachments on the surface of a stressed plate, and they fail by fatigue crack growth from the weld toe. In 7.3, we learnt that the main reasons for this poor fatigue performance were the stress concentrations (and intrusions) at the weld toe and the tensile residual stress in the region of the weld toe introduced by welding. There are two broad groups of improvement techniques; in one, measures are taken to reduce the weld toe stress concentration factor (and remove the weld toe intrusions); in the second, treatments are applied to modify the residual stress distribution and make the residual stresses near the weld toe compressive rather than tensile.

Post weld improvement techniques are now well established and methods for their application and ensuing benefits are described in fatigue design codes such as the IIW recommendations in report IIW-2142-110, BS 7608 and DNV RP C203. However, experience shows that it is not good practice to rely on fatigue improvement techniques at the design stage. It is preferable to take steps before welding, as described in the previous section, and to keep the possibility of applying a post-weld improvement technique in reserve should there be a very late change in the anticipated service requirement. Post-weld fatigue improvement techniques are particularly valuable when developing a procedure for the repair welding of a fatigue crack (see 13.8). For all fatigue improvement techniques, it is recommended that a formal procedure is developed and followed, and that operators are fully trained in carrying out the procedure.

13.3.1 Reducing the weld toe stress concentration factor

Grinding and remelting the weld toe can be used in order to reduce the stress concentration factor.

13.3.1.1 Weld toe grinding

Local grinding of the weld is probably the most common and most efficient of all the improvement techniques. A rotating burr (with a typical diameter of 12 mm, see Figure 13.1) is used to grind the entire length of a critical weld toe. As shown in Figure 13.2, the depth of the grinding should be 0.5 mm below any visible undercut.

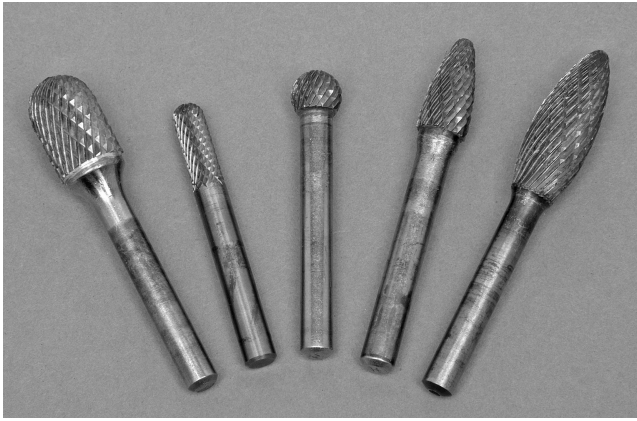


Figure 13.1 Burr grinding tools (© TWI).

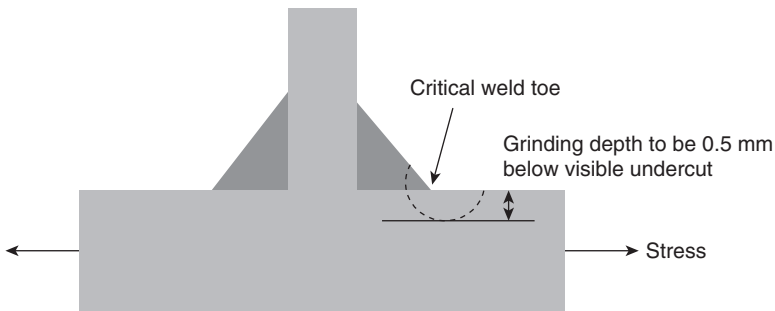


Figure 13.2 Weld toe grinding, sketched in cross section.

By carrying out grinding in this way, two objectives are met: the joint stress concentration is reduced and the weld toe intrusions are removed. There may be some concern that grinding below the plate surface reduces the thickness of the plate and, hence, increases the stress. However, providing the plate is thicker than 10 mm, the benefits of reducing the stress concentration more than outweigh the drawbacks of reducing the load-bearing thickness.

The fatigue design codes differ in detail regarding the benefit that can be achieved by weld toe grinding. As a rough guide, however, it could be assumed that the allowable stress range is increased by approximately 30%, which corresponds to a doubling of fatigue life.

As toe grinding relies on the introduction of a smooth profile, if the joint is exposed to a corrosive environment such as seawater, surface attack and pitting reintroduce stress raisers and result in the loss of most of the benefit. Similarly, attempting to achieve the same toe ground profile with a disc grinder instead of a burr grinder introduces grinding marks and scratches along the axis of the weld, rather than across

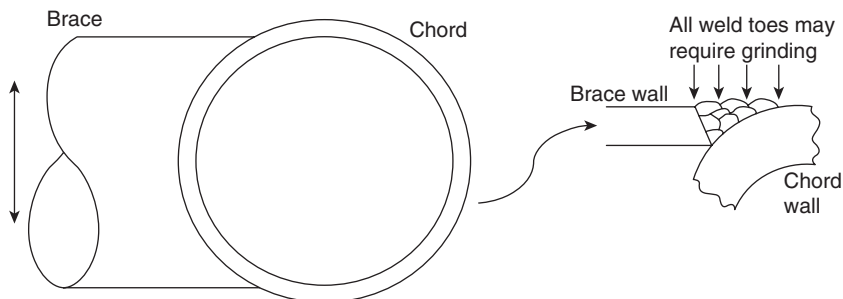


Figure 13.3 Brace to chord tubular joint.

the weld, thus also reducing the benefit of the fatigue life improvement. Indeed, the level of control possible with a disc grinder is far below the skill that can be applied to burr grinding of weld toes.

For critical tubular joints, where the brace and chord diameters are similar, fatigue cracks may initiate at the weld toe between the weld and the chord and at inter-run weld toes as all these locations experience very similar stress concentration factors, Figure 13.3. Therefore, it may not just be one weld toe location but some or all the weld toes across the weld cap that need to be treated.

For butt welds, grinding the weld cap flush with the parent material will significantly improve fatigue performance.

13.3.1.2 Weld toe remelting

Weld toe remelting both removes the weld toe intrusions and reduces the stress concentration at the toe. Both tungsten inert gas welding (TIG) and plasma welding can be used, because these processes are assumed not to result in further intrusions being formed at the remelted weld toes. No filler material is added, and low heat input weld passes are made along each weld toe of concern. The intention is to create small, localised remelted regions around the weld toe, rather than to modify the whole joint. Experience shows that the optimum shape of the remelted zone is achieved when the arc strikes the plate approximately 1 mm from the weld toe, as shown in Figure 13.4. The benefits of weld toe remelting are similar to those of weld toe grinding. Weld toe remelting, however, has not been used in many practical applications. It involves an additional welding operation often using a welding process that has not been used during the fabrication of the joint. The technique also requires skilled welders to maintain the correct heat input and positioning of the arc throughout the manual weld.

13.3.2 Introducing a compressive residual stress

The principle of introducing compressive residual stresses may be understood by using a similar model to that described earlier in 7.3.4 of this book. After welding,

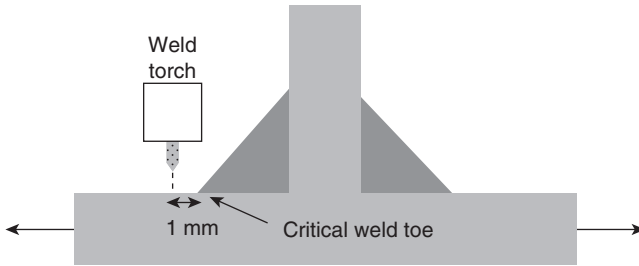


Figure 13.4 Weld toe remelting.

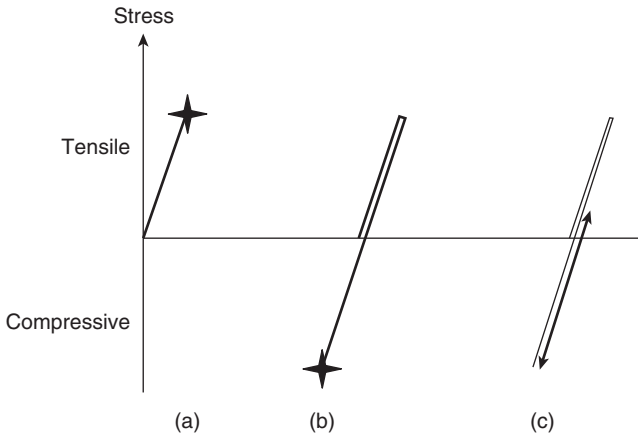


Figure 13.5 Principle of residual stress modification techniques, showing the stress cycle: (a) as welded, (b) under compressive stress and (c) the fatigue cycle.

the material at the weld toe is subjected to a tensile stress and is at the yield point of the stress-strain curve. If a compressive stress is applied, the local stress is reduced, eventually becoming compressive, even reaching yield magnitude in compression. An imposed cyclic stress causes the material to experience a stress range based on its new, compressive residual stress level. This is illustrated in Figure 13.5. Under these circumstances, the cyclic stresses experienced by the weld toe range from yield in compression to increasingly tensile stresses. At least part of the stress cycle (and perhaps all of it) is compressive at the weld toe. Cracks can only grow under a tensile stress; compressive stresses do not result in crack propagation. Therefore, from a fatigue standpoint, compressive stress ranges are not as damaging as tensile stress ranges, and, hence, an improvement in the fatigue life is achieved. Such modifications of the residual stresses at the weld toe are made by local plastic deformation in the vicinity of the weld toe.

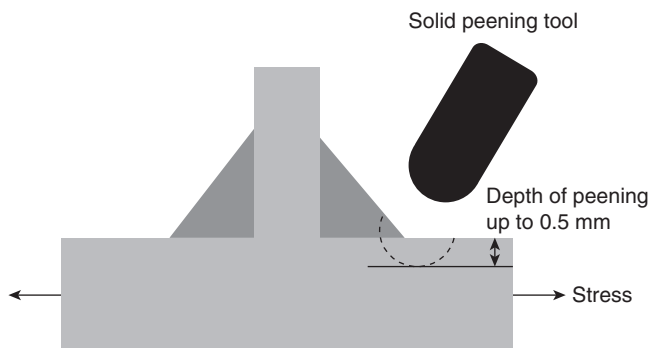


Figure 13.6 Hammer peening.

13.3.2.1 Hammer peening

A solid tool, typically with a 3–9 mm diameter radius at the tip, is directed at the weld toe and a pneumatic tool is used to generate a hammering action. Generally, a depth of peening of approximately 0.5 mm is required and four passes of the peening tool along the weld toe region are required to achieve this depth, as illustrated in Figure 13.6.

For partly convex fillet welds, it may be necessary first to lightly grind the weld toe to provide a guide for the peening tool along the weld line. A disadvantage of hammer peening is that the original weld toe is not removed. Furthermore, the weld toe is intensively deformed plastically and may give rise to features similar to a cold lap. When inspected after some years in service, magnetic particle inspection (MPI) may give rise to an indication that is difficult to interpret because it could be the original cold lap or a crack that has formed in service. If local light grinding removes the indication then it is likely that it resulted from the original cold lap.

The fatigue standards give differing detailed guidance on the benefit of hammer peening. It is at least as good as weld toe grinding, i.e. giving rise to a doubling of fatigue strength, and may even be slightly better.

13.3.2.2 Other peening techniques

Other peening techniques include:

- Needle peening, in which the solid tool is replaced by a bunch of needles hammered into the weld toe using a pneumatic tool. The advantages of this are that it is possible to peen over a larger area, but it is not as precise as hammer peening.
- Ultrasonic peening, in which a solid tool is used but the hammering action is achieved using a piezoelectrically excited hammer. Ultrasonic peening has several advantages; it is much quieter than the other peening techniques and the tool is much lighter and easier to manipulate (Figure 13.7).
- Shot peening, in which the joint is typically enclosed in a chamber and shot particles (which range in size from 50 μm to 5 mm in diameter) are fired at the surface using a gas jet. The



Figure 13.7 Ultrasonic peening around the weld toes of a fillet welded attachment to a fatigue test specimen (© TWI).

force is sufficient to cause plastic deformation of the surface. Shot peening is routinely used for bulk surface treatment of critical components such as gas-turbine blades but it has not yet been widely applied to improve the fatigue performance of welded joints.

13.4 Other fatigue considerations

13.4.1 *High-strength steels*

In Section 7.2.2 of this book, we emphasised that the fatigue strength of welded joints is independent of the yield strength of the steel. Using a higher strength steel in itself therefore confers no benefit. There is, however, some evidence that the benefits of applying a post-weld improvement technique are greater in joints made from higher strength steel. This is believed to be because the smooth surface generated by grinding introduces a crack-initiation phase that is longer in high-strength steels.

In the peening techniques, it may be possible to introduce even greater compressive residual stress owing to the higher material yield strength; this in turn gives rise to an even greater proportion of the applied cyclic stress range being experienced as compressive.

13.4.2 Post-weld heat treatment

Post-weld heat treatment (PWHT) reduces the levels of tensile residual stresses in a joint. It does not reduce these levels to zero, however, and, even in a very well controlled thermal cycle, the levels of final residual stress are unlikely to be much below 30% of the material yield strength. If a tensile residual stress of, say, 100 MPa, remains after PWHT, almost all applied fatigue cycles (assuming these are typically less than 100 MPa in stress range) still result in entirely tensile stresses at the weld toe and are fully damaging.

As the whole applied stress cycle is almost certainly still tensile, fatigue design codes do not recognise any benefit in terms of fatigue life and require the same design approach to be used for as-welded and PWHT joints. However, PWHT does not make the fatigue performance of the joint any worse, and it may be carried out for other reasons such as dimensional control, improving fracture performance, or prevention of hydrogen cracking.

13.5 Improving fracture performance

Although there are no fracture improvement techniques analogous to the post-weld fatigue improvement techniques, there are some general principles that help to avoid both brittle and ductile fracture. Before fabrication, the design principle to avoid ductile overload is primarily to limit the loading to below the material's yield strength (i.e. the design stress). The main concern is to identify the appropriate strength upon which to base the design. This is the lowest strength, possibly because of the statistical variation in material tensile properties or the reduced strength experienced at elevated temperature service.

Although restricting the applied stress helps to avoid brittle fracture, there are some additional considerations that affect the risk of brittle fracture. Ensuring sufficiently high fracture toughness is often based on assumptions from Charpy test data, but specifically carrying out fracture toughness testing is an alternative (see 9.5 and 9.6). The fracture toughness of the weld is often dependent on the microstructures present, with fine-grain structures giving the best fracture toughness. The microstructures present in welded structures are not usually significantly altered by heat treatment after welding. A normalising heat treatment may result in grain refinement, but the softening that occurs during this heat treatment makes this impractical for fabricated structures, and it is only really used for plain steel products.

PWHT does, however, reduce the residual stresses in a welded structure, and it is this effect that means that PWHT is probably the most effective method to apply after

welding to improve the fracture performance of the structure. The reduction in the level of residual stress can be estimated, based on the proportion of the yield strength of the steel (down to typically 30% of the yield strength). The considerable improvement in fracture performance between as-welded and PWHT joints can be demonstrated in fitness-for-service assessments.

13.6 Using this book: repair of fatigue cracks

To illustrate how to bring together many of the themes that have been presented in this book we consider the repair of cracks that have been found in service, such as fatigue cracks.

We can assume that the crack occurred in service if we know how the component was fabricated (Chapter 3) and that the quality assurance during and after welding meant that the installation met appropriate flaw tolerance standards. In order to detect the crack in the first place, it may necessary to apply a number of different inspection techniques (Chapter 10), because those best able to detect the presence of the defect may not be the best way to measure its size and dimensions. In-service cracking may have occurred owing to mechanisms such as stress corrosion cracking, creep, corrosion or fatigue, so the nature of the defect must be determined, from information about the defect orientation and location, and knowledge of the service conditions experienced. A fatigue crack is flat and at 90° to the applied cyclic load (Chapter 8).

In this scenario, it is assumed that final failure has not yet occurred. However, the next step is to determine how close to failure the cracked component is, by performing a fracture mechanics calculation. The assessment determines the maximum flaw size before brittle fracture can occur (Chapters 5 and 11), and calculates the minimum number of fatigue cycles the crack can undergo until it grows to that size (Chapter 12).

In order to perform these fitness-for-service calculations for the crack, it is necessary to know the mechanical properties of the weld and parent metal in the structure (Chapter 9) and, as well as knowing the size of the flaw, to understand the static and cyclic loading in the region of the crack (Chapters 4 and 6). If the crack can be shown to be safe, at least for a limited amount of future service, then no repair action is needed straight away.

If a repair is needed immediately, for shallow cracks it may be possible to grind the crack out completely and blend the excavated region to reduce stress concentration. The remaining wall thickness may be sufficient without any additional welding.

If this approach is not suitable, the whole crack must be excavated by grinding. The groove should be inspected to ensure that no crack remains at the bottom of the groove that could continue propagating after 'repair'. Once the fatigue crack has been satisfactorily removed, then the groove is filled using repair welding with a qualified weld procedure to make good the wall thickness. After the welding and final inspection, it is necessary to apply a fatigue improvement technique, such as those described in this chapter, in order to prevent further fatigue cracks developing as they did in the original joint.

13.7 Conclusions

Some weld designs, such as fillet welds and welded attachments, exhibit particularly poor fatigue performance in the as-welded condition. There are a number of measures that can be implemented at the design stage to reduce the effect of the joint design on the fatigue performance and to ensure good fracture performance. However, even after welding, it is possible to improve fatigue life by applying techniques that reduce the stress concentration effect of the weld toes, or induce a compressive residual stress at the weld toe. Where brittle fracture is the main concern, applying PWHT after welding confers the most benefit.

The example of repairing a fatigue crack detected in service shows how this book provides an understanding of the welding engineering concepts necessary for the avoidance of fatigue and fracture in welded structures and components.

13.8 Sources of further information and advice

BS 7608 (1993) *Code of practice for fatigue design and assessment of steel structures*, British Standards Institution.

DNV-RP-C203 (2011) *Fatigue design of offshore steel structures*, Det Norske Veritas.

Haagensen, P. J. and Maddox S. J. (2013) *IIW recommendations on methods for improving the fatigue strength of welded joints: IIW-2142-110*, Woodhead Publishing Ltd, ISBN-10: 1782420649, ISBN-13: 978-1782420644.

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