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ICE manual of construction materials

Volume **2**
Metals and alloys; Polymers; Polymer fibre composites in civil engineering; Timber; Glass; Non-conventional materials; Appendices

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Section 5: Metals and alloys

Section editor: Wei Sha

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Chapter 38

Metals and alloys: an introduction

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This chapter introduces metals and alloys as a type of construction material, including discussions of their background and use in construction, their mechanical and physical properties, microstructural factors that influence properties, and recycling and sustainability issues associated with metals and alloys. It references chapters in the Metals and Alloys Section of the *Manual of Construction Materials* and gives a brief overview of the coverage in order to point readers towards those chapters. The chapter serves as a starting point to a practitioner researching the properties and usefulness of metals and alloys to construction applications.

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Metals and alloys as construction materials

In terms of volume or weight, metals and alloys as construction materials are well behind concrete. However, their strength, ductility, toughness, surface properties, weldability, electrical and thermal conductivity, and many other properties unique to this group of materials, make metals and alloys indispensible materials for construction. These materials are used in houses and other buildings, structural steelwork including buildings, bridges, offshore structures and piling, road structures, and does not require the extensive machinery necessary for completing any major construction project, for example cranes, trucks and concrete mixers.

A variety of metals are used. The predominant type is, of course, the ferrous metals, i.e. iron and steel. Within this type, there are cast iron, wrought iron, carbon steels and alloy steels, differentiated mainly by their composition. Among them, carbon steels form the basis of structural steelwork as well as concrete reinforcing bars, and thus are used in the largest quantity among all types of metals. Although metals were used extensively from ancient times, as indicated by history terminologies such as the Bronze and Iron Age, the scale of their use was dramatically increased with the modern steelmaking technology. Although the development of steelmaking technology has stabilised in the last few decades, the processing techniques for steels as well as for iron have continued to develop and improve to great effect, including thermomechanical processing, heat treatment, and surface engineering.

Among the non-ferrous metals, aluminium, zinc, lead, copper and tin are traditionally widely used. Less widely used are nickel and chromium. There are also new metallic materials for construction, for example titanium for cladding purposes. Although far from widely used, titanium offers an attractive alternative, due mainly to its aesthetic features and corrosion resistance. The development of non-ferrous metals and alloys has been more rapid than that of iron and steel, although this is mainly to catch up with rather than replace the ferrous metals.

Microstructure and properties of metals and alloys

Microstructural defects and their relations to strength and ductility

The properties of metals and alloys which are unique to this group of materials, as opposed to say ceramics and polymers are due to their microstructure, starting from the small, atomic scale of metallic bonds, up to their grain and phase structures, usually in the micrometre scale, hence the term microstructure. In metals, there are free electrons shared by many atoms, as against the ionic bond or covalent bond in ceramics and polymers. Such free electrons are the cause of the metals' electrical and thermal conductivity. This unique type of bonding also determines the high strength and ductility of metals and alloys.

In materials science, materials are broadly divided into structural materials and functional materials. The application of the first group is based on the material strength, while for the second group it is based on electrical and electronic, magnetic and optical properties. The most widely used properties of metals and alloys for construction purposes are their strength and ductility, for the so-called structural materials. In physical metallurgy and materials science, the strength and ductility are explained mainly with two microstructural terms, namely grain structures and dislocation structures.

Microscopically, with the exception of amorphous materials not normally used in construction, a piece of metal is packed with small crystals, although the exterior of the metal does not normally show features of crystal that we normally associate with geological minerals. The

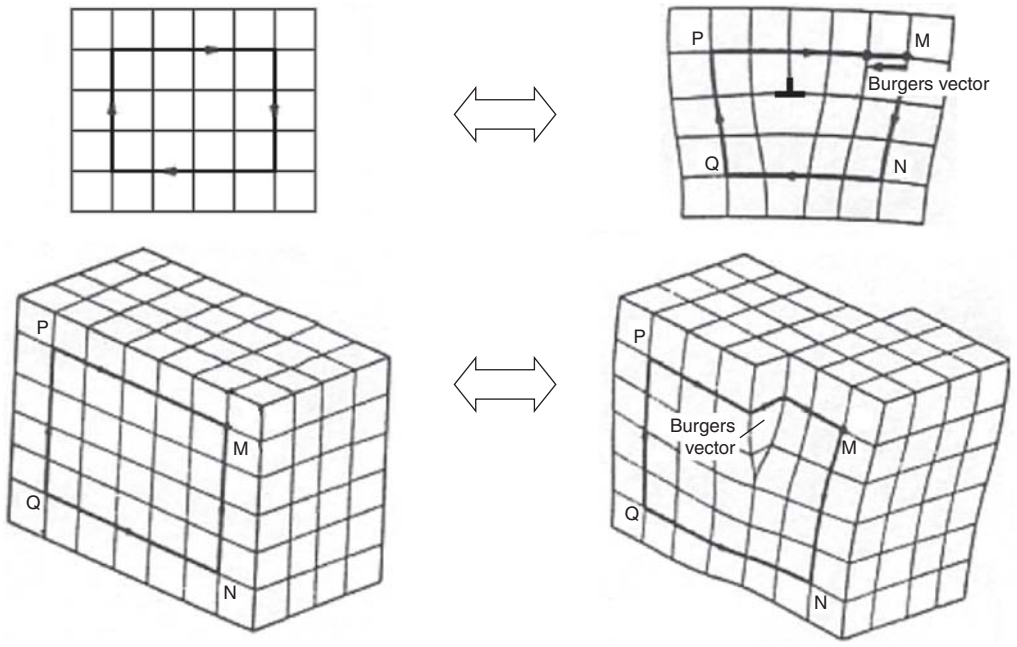

Figure 1 The lattice arrangements of edge (top right) and screw (bottom right) dislocations compared to perfect crystal structures

reason why we cannot see the crystals (or even signs of them) with the naked eye is because they are very tiny, usually around the scale of 10^{-6} m (one micrometre) but they could be manipulated by special treatment to down to 10^{-9} m (one nanometre). Recent research has resulted in the development of nanostructures in bulk metallic materials (Bhadeshia, 2005).

These tiny crystals are packed densely with no gaps between them, and are only separated by grain boundaries. The grain boundaries have significant effects on metal properties such as strength, ductility, atom diffusion and conductivity.

For the perfect crystal structure to deform plastically, extremely large forces are required, in order for the atomic planes to slide against each other. Imagine even the force required to pull from one end and slide a large carpet on a floor, in one single movement. The friction force between the carpet and the floor is huge. The sliding of atomic planes has similar difficulties, only a million times harder because of the tight bonding between atoms next to each other in adjacent atomic planes. In real metals, however, there is a common type of defect called dislocations (**Figure 1**) that completely change the mechanism of plastic deformation.

Dislocations appear in large numbers in metals and alloys, some formed naturally during cooling after melting during the alloy manufacture, and some formed during deformation processing such as rolling and forging, and some even formed during the plastic deformation process which is facilitated by them in the first place. With dislocations (but not too many because then they will tangle up and be hard to move), plastic deformation becomes easier because, instead of moving the whole crystal planes in one go, it is much easier to push each dislocation to move. This is very similar to moving carpets by first forming a kink, and then pushing the kink from one end of the carpet to another. Worms move in the same way, too.

The deformation mechanism through the movement of dislocations is the reason why metals have good ductility. Grain boundaries, on the other hand, limit the movement of dislocations because it is not always easy for dislocations to traverse across them. By controlling the quantity and size of the grain boundaries and dislocations, virtually unlimited range of property combinations can be achieved.

A grain boundary caused by the discontinuity of crystals is a 2D defect, and a dislocation caused by atomic plane misalignment is a 1D defect. In addition to 1D and 2D defects, there are 0D defects, the so-called point defects, usually in the form of vacancies, i.e. lattice spots, but not occupied by any atom. The vacancies determine the rate of diffusion because metal atoms diffuse by jumping into an adjacent vacancy. There are 3D defects, too, in the form of inclusion and precipitation for example, which weakens or strengthens the metallic materials depending on their characteristics. As it transpires, atomic and micro-scale defects in metals are far more important and useful than the perfect crystal structure.

The physical metallurgy and materials science of metals and alloys are described in many metallurgy textbooks. A

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comprehensive source of information for metals is the *ASM Handbook* series published by ASM International. For free online information, the Wikipedia (http://en.wikipedia.org) contains many relevant articles, although they may not be as authoritative.

Corrosion and fire resistance

The most commonly used metals, i.e. structural steels, are one order of magnitude stronger than concrete, and even much stronger than other construction materials such as timber and bitumen. However, some other properties of metals and alloys also contribute, or sometimes limit their use in the construction industry.

One such property is corrosion. Most metals react with oxygen, i.e. they are oxidised in air. When moisture is present, e.g. under water or in a high humidity environment, the combined effect of water and oxygen can cause corrosion. This is a major problem for structural steels, and a high proportion of construction cost of structural steelwork is on corrosion prevention. However, other metals, such as aluminium, titanium and stainless steel, have means of overcoming the corrosion problem, by forming a thin and stable protective layer on their surface. The prevention or minimisation of corrosion does not always rely on using non-metals which do not have corrosion problems themselves, such as polymer-based paints. One of the most effective anti-corrosion measures is to use zinc and, to a lesser extent, aluminium, two of the most active metals. There is a wide variety of protection techniques based on the galvanising action between zinc/aluminium and steels, because zinc/aluminium, when in contact with steels, will corrode first and in fact eliminate the corrosion of steels. Such techniques include zinc (alloy) galvanising, metal spraying and painting containing zinc or aluminium. Metals can be used to protect other metals.

Another consideration, usually to the disadvantage of using metals in construction, in particular structural steels, is related to the properties of metals at high temperatures, for example in a fire. The reason for the usually relatively poor metal performance at elevated temperatures is related to its metallic bond. The high thermal conductivity is as important as the loss of strength itself, because it permits heat to reach and spread over metal parts quickly. This is a particular issue for structural steelwork, the issue of its fire resistance. Although the concrete strength would have dropped even more heavily at a given temperature than steel, the low thermal conductivity and the bulkiness of the concrete mean that a concrete structure does not usually have any fire resistance problem, simply because the heat does not reach the inside of concrete. On the other hand, structural steelwork, with some exceptions of, for example, open car park structures, will always need fire protection, which adds a significant amount of cost. In recent years, there have been large advances in fire engineering research and development, resulting in some reduction of fire protection cost in structural steelwork.

Sustainability and recycling

Metals and alloys are champions in the race towards maximum sustainability and recycling. In principle, all metals and alloys can be recycled, usually up towards 100%, although in practice, this may be achieved only with considerable cost.

The fundamental reason for the excellent recyclability of this class of materials is, in the author's view, that metals originally appear in earth not in their pure form but in compound form with, for instance, oxygen, i.e. oxides. These are ores. Metals need to be extracted from the ores, i.e. separated from the compounded or mixed elements, which is a long and costly process. After a metal is used, if it has changed from its pure state, it may be contaminated, or mixed or compounded with other materials, it may be corroded or rusted and it may be oxidised. However, in principle, the goal of returning such used, or non-pure metal to its pure form (i.e. from old to new) is fundamentally no different from extracting the metal from ore. The process may be very different however, but it is in many cases cheaper. This, plus the pressure from the vanishing resources that we are facing, make the idea of recycling metals very attractive.

One major example is the use of the electric arc furnace for steel making. This process uses scrap steel as its raw material, and produces high-quality alloy steels, for example stainless steel, a product used heavily for construction. The process has been in use for more than 100 years, long, long before the word 'sustainability' was invented. This was because metallurgists and steel users knew the recyclability of steels, and the value of it. This same example also illustrates the cost factor mentioned above, because an electric arc furnace operates with a high consumption of electricity, and the cost of running it is higher than the other route of steel making, the basic oxygen furnace, which does not use scrap steel as the main source of raw material. Despite the cost factor, a higher and higher fraction of metals and alloys is being recycled, due to environmental regulations and sustainability issues and concerns.

Section contents and authors

This section of the *ICE Manual of Construction Materials* gives detailed information on the class of materials of metals and alloys. Before dealing with individual types, the section starts with describing the nature and behaviour of alloys, which essentially covers basic metallurgy, needed for understanding and appreciating the properties of metals

and alloys. This chapter is not about specific types of metals, it intends to give the basic metallurgical theory that is applicable to all metals and alloys.

The chapters that follow describe the most common types of metals and alloys used in construction, including ferrous metals, aluminium, copper and zinc. In addition, an up-and-coming metal not previously used in construction, titanium, is discussed in Chapter 44. The focus is on the properties and uses of these metals, so that the reader can apply the knowledge straight to the practical use of them in the design and maintenance of structures and construction projects.

Traditionally and for the foreseeable future, in terms of tonnage, structural steels are dominant among metals and alloys used in construction, so the final chapter concentrates on the application and design issues of structural steels. The aim of this chapter is for the reader to gain an advanced knowledge of these materials, and be able to apply this knowledge directly in civil engineering construction.

I am extremely proud to say that we have got a top, distinguished team of British authors for these chapters and topics, each a world authority in his own field. Chapter 39, 'The nature and behaviour of alloys', is written by Dr Joseph Robson, a Senior Lecturer in Physical Metallurgy at the University of Manchester. Among other professional achievements and credentials, he is an associate editor of *Materials Characterization*, an international journal published by the International Metallographic Society on materials structure and behaviour.

As Chapter 40 is concerned with different types of metals, we have several authors, writing about each type of metal and its alloys. The chapters on ferrous metals and zinc are written by Arthur Lyons. His textbook, *Materials for Architects & Builders*, is widely used by universities for teaching of their Construction Materials module, including that run at my own university, coordinated by myself. However, for the *Manual*, Dr Lyons is able to elaborate to a much more advanced level compared with the undergraduate textbook. The chapters on aluminium and copper are written by respective top specialists in these metals, namely David Harris of Aluminium Advocates and Peter Webster of the Copper Development Association. Neil Lowrie, a technologist at NAMTEC, has contributed the chapter on titanium (NAMTEC is the National Metals Technology Centre in the UK).

The last chapter of the Metals and Alloys Section is written by Mark Lawson, a most familiar name in structural steels in the UK and beyond, for his influential work at the Steel Construction Institute (SCI). Technical publications on structural steel design produced by SCI under his direction are widely used by consulting engineers as well as in the relevant professional and higher education sectors. SCI also has a strong influence in British Standards and now Eurocodes for structural steelwork.

Each of these authors is a fellow or member of the relevant professional bodies in the UK.

I am therefore confident that this section of the *Manual of Construction Materials* forms a most authoritative guide of metals and alloys for construction practitioners.

Reference

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Chapter 39

The nature and behaviour of alloys

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The performance of metal alloys is understood by studying the relationships between composition, processing, structure and properties. Knowledge and control of structure on the microscopic scale is critical in optimising macroscopic performance, producing the versatile range of alloys available today.

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Introduction

Metal alloys form one of the most widely used classes of materials in engineering applications. Metals are rarely used in their pure elemental form, but are combined into mixtures of elements known as alloys. Alloying is the deliberate addition of extra elements to a pure metal to improve some aspect of its properties. In addition to deliberate alloying elements, all commercial metals also contain impurity elements that are uneconomical to remove during refinement and processing. Alloying can produce large improvements in properties. For example, steels and aluminium alloys can have strength levels that are over an order of magnitude greater than the base metal (iron and aluminium respectively) in pure form.

The engineering properties of metals, such as strength, toughness and corrosion resistance, are controlled by the structure of the metal on the nano- (10^{-9} m) and micro- (10^{-6} m) structural scale. In particular, it is through manipulation of alloy structure at the micro-scale (the microstructure) that required properties are obtained. The metallurgist therefore has to consider the relationships between alloy composition, processing, microstructure and properties. This chapter will introduce the most important of these relationships for commonly used engineering alloys.

Microstructure of metals and alloys

Structural alloys are all crystalline at an atomic scale; that is, their atoms are arranged in a regular, ordered way. The individual crystals (which are usually referred to as grains) are typically less than a millimetre in size, so that bulk metals are an aggregate of many millions of grains (i.e. polycrystalline).

The arrangement of atoms in a crystalline material can be defined by its 'unit cell'. The unit cell is the minimum volume of material that fully characterises the crystal structure and symmetry. The structure of any (perfect) crystal can be generated by repetition of its unit cell in three dimensions. The metals that form the basis of the common engineering alloys have crystal structures that can be characterised by one of three unit cells. **Figure 1** shows how the atoms are arranged in each of these cells.

From an engineering perspective, the crystal structure of alloys is important, since it is critical in determining many of the macroscopic properties. For example, aluminium is easily deformed at room temperature without fracturing, whereas magnesium will crack if deformed under the same conditions. This can be largely attributed to the hexagonal crystal structure of magnesium compared with the face-centred cubic crystal structure of aluminium, as discussed later.

Phase transformations

In many alloys, several different crystal structures (allotropes) are possible. The most stable crystal structure will then depend on the external conditions (such as temperature and pressure). Alloying additions can also influence the relative stability of the different allotropes. If the conditions are changed so that the most stable crystal structure changes, the atoms will attempt to rearrange from the old to the new stable crystal structure. The different crystalline forms that an alloy can exist in are referred to as different phases, and so the change from one crystal structure to another is known as a *phase transformation*. Phase transformations are exploited in the processing of alloys to obtain the required microstructure. The best-known example is

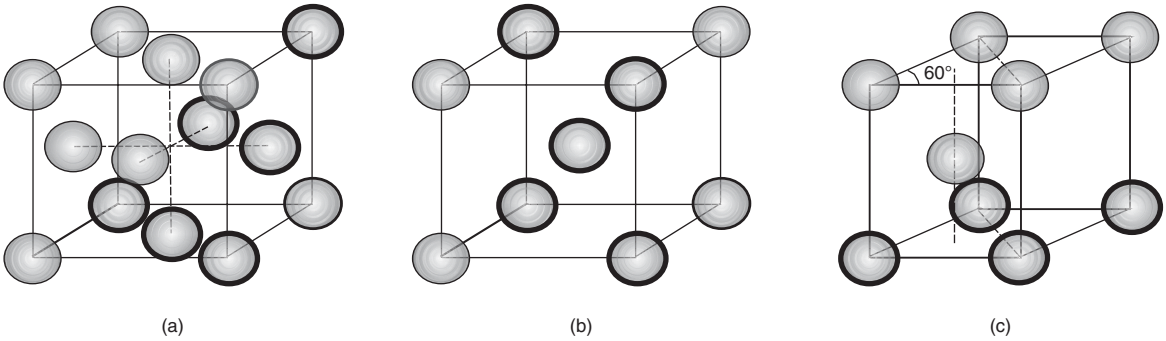

Figure 1 Unit cells showing the arrangement of atoms in (a) face-centred cubic, (b) body-centred cubic and (c) hexagonal close-packed crystal structures. The atoms outlined in bold define one of the planes of closest atomic packing in each structure

the phase transformation that occurs in iron and steel (steels are based on the iron–carbon system, with additional alloying elements). At room temperature, the most stable crystal structure for iron is body-centred cubic (BCC). This phase is called ferrite. When heated above 910°C (1183 K), the face-centred cubic (FCC) structure becomes more stable, and a phase transformation occurs. The FCC iron phase is called austenite.

The presence of additional elements, as in steel, adds to the complexity of the phase transformations that can occur. Steels are often processed with the iron in the high-temperature (austenite) form before being cooled to room temperature, where the ferrite phase is stable. Austenite has a higher solubility for carbon than ferrite (approximately 100 times more carbon can be dissolved in austenite than ferrite, by weight). This means that for iron to form stable ferrite on cooling, carbon must be rejected from the transforming austenite. This carbon forms a new phase, which in simple (plain carbon) steels has the chemical formulae Fe_3C, and is called cementite. If other alloying elements are present, they can also form new phases with the excess carbon, known as carbides.

If cooling is very rapid (such as obtained if steel is quenched into water) then there is usually insufficient time for the movement of atoms that is required to re-arrange the austenite crystal structure to the two new crystal structures of ferrite and cementite. In this case, a series of different phase transformations is possible, forming phases that give different microstructures and properties. The ability to exploit the phase transformations from austenite to give a range of different structures is a great advantage of steels. Apart from iron/steel, other important industrial metals that transform from one crystal structure at high temperature to another at lower temperature include titanium and zirconium. The stable structure for both of these is hexagonal close packed (HCP) at room temperature, but BCC at high temperature.

Another type of phase transformation that is widely exploited in engineering alloys is particle precipitation. Particle precipitation refers to the formation of (usually) micron- or sub-micron-sized particles of a second phase in the initial (matrix) phase. Precipitation occurs because the solubility of alloying elements in the matrix phase decreases with decreasing temperature. When the solubility is exceeded, there will be a driving force for precipitate formation. Precipitation involves alloying elements leaving the matrix phase and forming particles of a different phase with distinct composition and structure. This process takes time, however, and precipitation can be suppressed by cooling very rapidly. In this case, excess alloying elements can be trapped in the matrix phase. These elements will then form precipitate particles over time, and heating can accelerate this process by enhancing atomic movement (diffusion). This method can be used to produce a fine and uniform distribution of precipitate particles that can provide a large strengthening effect (precipitation strengthening) (AluMATTER, 2007). Precipitation strengthening is exploited in many aluminium alloys, steels, and other alloy systems.

Defects in metals and alloys

The crystals that are generated by stacking together unit cells, as described in the previous section, are perfect; there are no irregularities in such a structure, and each part of the crystal is identical to every other part. In reality, real crystals are not perfect but contain defects. These defects play a crucial role in controlling the properties of metals and alloys.

Two types of defect are of particular importance in metals and alloys: vacancies and dislocations. A vacancy is a 'gap' in the crystal structure, where there is no atom where one would be expected (**Figure 2(a)**). Vacancies are important in the process of atomic diffusion (the movement of atoms within the crystal structure). Any one of the atoms that surround the vacancy can move by jumping into the vacant site, leaving a vacant site behind (the atom and vacancy swap positions). Another atom can then jump into the newly vacant site and so

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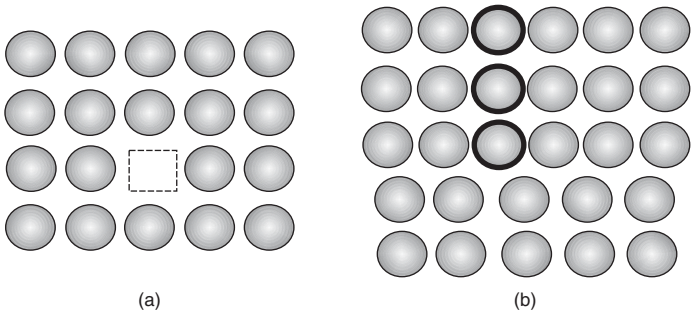

Figure 2 Schematic showing (a) a vacancy in a crystal structure (b) an edge dislocation: the atoms outlined in bold form the extra half plane, and the line where this ends inside the crystal defines the dislocation line

on. A vacancy is one type of *point defect* since it is localised to a single point in the crystal structure.

Dislocations are an example of a *line defect*, creating a disruption to the crystal lattice around a line rather than a single point. **Figure 2(b)** shows an example of one type of dislocation, an *edge dislocation*. An edge dislocation is caused by the presence of an extra portion of a plane of atoms that terminates within the crystal (commonly referred to as an extra 'half plane'). The atoms that form the extra half plane are circled in blue in **Figure 2(b)**. Other dislocation types, such as screw and mixed dislocations are also possible (Hull and Bacon, 2001). Dislocations play a key role in the deformation of metals and alloys; deformation, usually occurs primarily by dislocation motion.

Plastic deformation of metals and alloys

It is possible to prepare and deform large single crystals of metals. For example, consider a single crystal specimen pulled in tension. At first, deformation occurs elastically and, if the load is removed, the specimen will return to its original dimensions. However, above a certain applied stress (the yield stress) the specimen starts to permanently (plastically) deform. It is found that when the surface of a suitable chosen crystal is examined after such a test, it is characterised by a series of bands, close to 45° to the axis along which the crystal was pulled, and these bands form steps at the surface of the crystal (**Figure 3(a)** shows these *slip bands* in a single crystal of cadmium after deformation). Furthermore, it is found that the planes defined by the slip bands (slip planes) correspond to the planes of closest atomic packing in the crystal structure. For certain crystal structures and orientations, it may be that none of the planes of closest atomic packing lie close to 45° to the axis along which the tensile load is applied. In such cases, it is often found that failure will occur by brittle fracture, rather than progressive plastic deformation.

Several important conclusions can be drawn from these observations. First, deformation occurs by shearing of the crystal along well-defined slip planes. This is true even if a purely tensile or compressive load is applied, since such a load will generate a shear stress (which is a maximum at 45° to the axis along which the tensile load is applied – the *tensile axis*). Second, deformation occurs only on certain well-defined planes in the crystal structure and in certain well-defined directions. These directions and planes are known as *slip directions* and *slip planes* respectively. It is found that the slip planes usually correspond to the planes on which the atoms in the crystal are most closely packed together, with the slip directions corresponding to the directions along which the atoms are in closest contact.

Figure 3(b) shows four unit cells of the FCC crystal structure. The slip directions and slip planes for this crystal structure are marked. It can be seen that there are four possible slip planes, each containing three possible slip

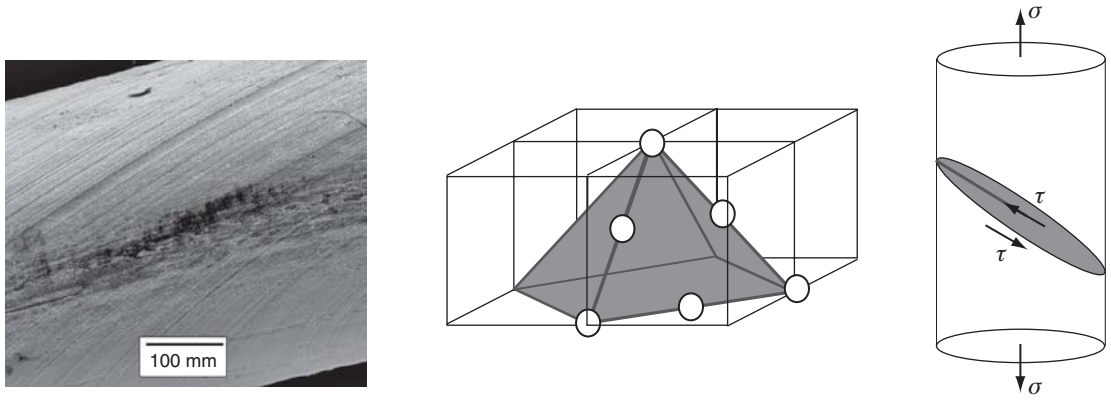

Figure 3 (a) A cadmium single crystal after deformation showing slip bands at the surface. (b) The close-packed planes and directions in the FCC crystal structure that define the 12 slip systems. (c) Schematic showing how an applied tensile stress σ produces a shear stress τ on a slip plane inclined close to 45° to the tensile axis

directions. This gives 12 combinations (4×3), and each combination is known as a slip system. Therefore, metals with FCC crystal structures have a total of 12 possible slip systems. Metals with BCC crystal structures also have at least 12 possible slip systems, whereas metals with HCP crystal structures may have as few as three slip systems. This is of practical importance, since the ease with which a metal can be deformed or shaped without cracking is strongly dependent on the number of available slip systems that can accommodate deformation.

Plastic deformation will initiate on a slip plane in a particular slip direction when the shear stress on that plane, resolved in the slip direction, exceeds a critical value. For a crystal pulled in tension (as shown in **Figure 3(c)**), the resolved shear stress will be a maximum on the slip planes that are oriented close to 45° to the tensile axis. Deformation will therefore take place by shearing on these planes.

How, on an atomic scale, does this shearing occur? It might be imagined that all of the atoms on one slip plane slide over all of the atoms on the slip plane below, since this would produce the observed deformation at the crystal surface. However, when a calculation is made of the theoretical critical shear stress required for this process, it is found to be several orders of magnitude greater than the critical shear stress measured, suggesting another mechanism must be operating.

In practice, it is much easier to produce slip by introducing and propagating a dislocation into the crystal structure, rather than sliding one whole atom plane over another. Then, rather than having to move all the atoms in the slip plane at the same time, disrupting the crystal structure across the whole crystal, only a local region of the crystal structure is disrupted at any one time. An animation illustrating this process can be found at (DoITPoMS, 2007).

The movement of one dislocation and its destruction at the surface of the crystal produce a deformation step of less than 0.5 nm for a typical metal, from which it is clear that movement of many millions of dislocations is needed to provide easily seen macroscopic deformation. Some of these dislocations will already exist in the crystal (since, as discussed, no real crystal is perfect). Others are generated during deformation (see Hull and Bacon, 2001 for more details).

The knowledge that deformation in metals occurs on an atomic level by movement of dislocations has important practical consequences, since it suggests that increasing a metal's resistance to plastic deformation (i.e. increasing strength) requires inhibition of dislocation movement. The mechanisms used to do this are discussed next.

Strengthening of metals and alloys

In many applications, yield strength is a critical mechanical property, since this is the maximum stress to which a material can be subjected without permanent plastic deformation. There are several ways that the yield strength of a metal can be increased, both by alloying additions and processing. Fundamentally, each of these methods works by making dislocation movement more difficult, thereby increasing the resolved shear stress that is required to initiate plastic deformation.

Grain size strengthening

So far, the deformation of a single crystal has been considered. In practice, bulk metals are nearly always polycrystalline and consist of aggregates of many crystals (grains) in different orientations. The boundaries between grains provide a barrier to dislocation motion. These boundaries therefore provide a strengthening effect and, the more grain boundary area there is per unit volume, the greater the strengthening effect. This means that fine-grained materials will be stronger than the same coarse-grained material, because a fine-grained material will have more grain boundary area to impede dislocation motion. The relationship between the grain size of an alloy and its yield strength is described by the *Hall–Petch* equation, which states that the yield strength is inversely proportional to the square root of the grain size (Callister, 2006).

A reduction in grain size can be achieved in practice by thermomechanical processing of the alloy (e.g. rolling it to produce sheet, or extruding it to produce rods or bars). It is a widely used strengthening mechanism since it does not rely on expensive alloying elements and, in addition, a grain size reduction also improves the toughness of many alloys (in contrast with the other strengthening mechanisms discussed here, where the increase in strength is usually accompanied by a decrease in toughness).

Solid solution strengthening

Some alloying elements can be dissolved into the crystal structure of the matrix phase forming a *solid solution*. Alloying elements can be accommodated into the crystal of the parent element in one of two ways. If the atomic radius of the alloying element is much smaller than the parent atomic radius, then the alloying element atoms can sit in the gaps (interstices) between the parent atoms. Such small atoms are commonly referred to as *interstitials* when in solution. Carbon atoms occupy the interstitial sites when dissolved in iron and this plays a key role in the metallurgy of steels.

More often, soluble alloying elements will not be small enough to be accommodated within gaps between parent atoms, in which case the alloying addition will be dissolved in the parent phase by replacing parent atoms in the crystal structure. Such elements are referred to as being *substitutional* when in solution.

Both interstitial and substitutional additions will create distortions (strains) in the crystal structure due to the

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mismatch in size between the parent and alloying element atomic radius. Dislocations also result in a local distortion (strain) because the atoms around the dislocation are not in their ideal positions. The strains due to dislocations will interact with the strains surrounding solute atoms because strains of opposite signs will try to cancel each other out. This creates an attraction between the dislocation and solute atom that must be broken for the dislocation to move, increasing strength. The effectiveness of solute elements in increasing yield strength increases with an increase in the misfit between the solute and parent atoms and also with the amount of the solute element that can be dissolved into the matrix crystal structure.

Strain hardening

Most metals have the useful property that they become stronger as they are plastically deformed. This effect is known as strain or work hardening, and it is often exploited in the strengthening of metals and alloys. The origins of the strain hardening effect can be traced to interactions between the dislocations that are generated during plastic deformation. As discussed, these dislocations locally distort the crystal structure and are thus surrounded by strain fields. As the number of dislocations increases during deformation, and the dislocations move, their spacing decreases until their strain fields start to interact with each other.

On average, the interaction between the strain fields of neighbouring dislocations leads to repulsive forces between the dislocations, with the result that an additional applied stress is required to overcome this repulsion. This results in an increase in strength. The effectiveness of strain hardening will depend on the amount of deformation imparted and also the type of alloy (some alloys show much greater strain hardening than others). Like grain size refinement, strain hardening is a widely used method to strengthen low-cost alloys, since it does not depend on the addition of expensive alloying elements.

Precipitation strengthening

The most potent strengthening mechanism that is exploited in the highest-strength alloys is precipitation strengthening (also known as *age hardening*). This strengthening mechanism involves forming fine (sub-micron) particles of a second crystallographic phase that are embedded in the matrix crystal. These particles are usually formed by a suitable precipitation heat treatment, as discussed previously. The precipitate particles increase strength by acting as barriers to dislocation motion. It is found that there is an optimum size and spacing of particles that gives the best strengthening effect (the optimum particle size is usually only a few nanometres). If the particles are too small, dislocations are able to cut through them, whereas if the particles are large and widely spaced dislocations can pass between the particles by bending (AluMATTER, 2007, Strengthening Mechanisms Module). The correct particle size is obtained by carefully controlling the alloy composition and heat treatment.

Other mechanical properties

The discussion so far has focused on the use of alloying elements and processing to increase the yield strength of alloys. While strength is usually a critical design property, there are other mechanical properties that must be considered, and may indeed be of overriding importance in some applications.

Stiffness

Stiffness (usually characterised by the Young's modulus) is the ability of a material to resist elastic deformation. Since most components are designed to operate within the elastic deformation regime (i.e. without plastic deformation) then stiffness is usually a key property. However, although different metals have widely varying stiffness values (e.g. the Young's modulus of tungsten is 400 GPa, that of magnesium is 45 GPa), processing and alloying elements added at typical levels do not usually greatly change the stiffness of the parent metal. Only in alloys that contain several phases, which have markedly different stiffness values, is it possible to obtain significant variations in stiffness by varying the proportion of the two phases.

Fracture toughness

Fracture toughness characterises the ability of a material to resist the propagation of cracks. Tiny cracks and defects are present in all commonly used materials. These cracks will lead to the concentration of stress at the crack tip; the critical stress concentration required to propagate the crack characterises the fracture toughness of the material. Materials with low fracture toughness, such as many ceramics, will tend to fail by rapid growth of intrinsic cracks before reaching the yield stress required for general plastic deformation. Most alloys have a higher fracture toughness than this, and will yield plastically before fracture.

Alloying and processing often have a large influence on fracture toughness of metals. The general trend observed is that any mechanism that increases the strength of an alloy concomitantly reduces its toughness. This is because once the yield stress is exceeded at the crack tip, plastic deformation is able to redistribute the stress and reduce the stress concentration. The exception to this trend is strengthening by grain size refinement, which usually increases both strength and fracture toughness. Fracture toughness is also degraded by the presence of a low-toughness brittle second phase, which can both act as sites for initial crack formation and as easy pathways for crack growth.

Fatigue

Fatigue occurs when a material is subject to a fluctuating stress, and can lead to failure after a period of time even if the maximum stress level experienced is considerably below the stress required for global plastic deformation (i.e. the yield stress). Fatigue is caused by the initiation and growth of a crack at a point of high stress concentration, which may be due to a design feature (e.g. a sharp corner) or a microstructural feature (e.g. large, hard particle).

Microstructure will influence both the initiation and propagation of fatigue cracks in alloys. Local defects in the microstructure such as voids or particles (e.g. inclusion particles formed by impurity elements) can act as stress concentrators and sites of crack initiation. Since fatigue is often initiated at surface scratches or defects, changing the surface properties can have a marked effect on fatigue resistance. Alloying additions and microstructure will also influence crack growth. Solute strengthening elements tend to increase fatigue properties in parallel with their effect on yield strength. Second phase particles and grain size have a more complex relationship with fatigue properties that depends both on alloy and the fatigue conditions, e.g. whether low-stress amplitude fatigue or high-stress amplitude fatigue (Callister, 2006). In general, alloys that rely on precipitates to obtain their strength generally have lower fatigue properties in relation to their yield strength than alloys strengthened by other mechanisms.

Creep

Creep describes the process by which alloys deform permanently when subject to a stress below the yield stress when held at elevated temperature. Creep in metals only becomes significant at temperatures greater than about $0.4\,T_m$, where T_m is the absolute melting temperature (K). It follows that metals with lower melting points (e.g. aluminium $T_m = 933\,K$, lead $T_m = 600\,K$) are more susceptible to creep than higher melting point metals (e.g. nickel $T_m = 1728\,K$, titanium $T_m = 1941\,K$). Creep resistance also depends on microstructural features. Grain boundaries accelerate creep deformation, so a fine grain size is undesirable for maximum creep resistance. Indeed, in the most demanding applications, such as high-performance jet engines, an entire component (e.g. a turbine blade) is made from a single crystal. Solid solution elements and precipitate particles can also enhance creep resistance, provided they are stable at elevated temperature.

Processing and forming of metals

The starting point for nearly all fabrication processes using alloys is molten metal. Alloying is most easily achieved by co-melting the base metal and alloying additions. Impurities can also be removed (e.g. by additions which form solid particles that sink to the bottom or float to the top of the melt, binding impurity elements).

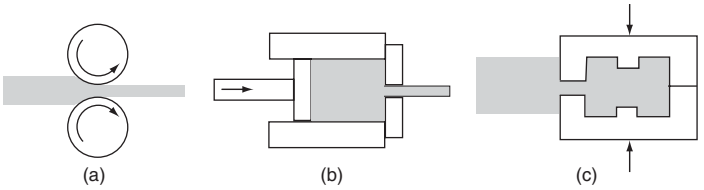

Figure 4 Thermomechanical processing methods used for the production of wrought alloy components: (a) rolling; (b) extrusion; and (c) forging

The molten metal is then solidified. In the case of *cast* components, solidification takes place in a mould that is close to the final shape of the product, so that once solidification is complete little further fabrication is undertaken (although heat treatment of the casting is common). The majority of metals, however, are predominantly used to make *wrought* products, which requires *thermomechanical processing* (TMP). In the fabrication of wrought products, casting is the first step in a process chain that includes deformation of the alloy. Deformation is initially carried out at elevated temperature, since this reduces the strength of the alloy, making it easier to work. Final deformation steps may be carried out cold. Typical processes used to make wrought components include rolling (to produce sheet, plate and beams), extrusion (to produce rods and bars), forging (to produce components with simple shapes but excellent properties), and drawing (to produce wire) (Dieter, 1989). These processes are shown in **Figure 4**.

Wrought products generally have better and more reliable properties than those of as-cast components. This is because it is very difficult to produce defect-free castings with the optimum microstructure (e.g. fine grain size, uniform distribution of strengthening particles). TMP allows greater control over the final microstructure and a reduction of defects such as pores, leading to the better properties exhibited by wrought products.

The microstructural changes that occur during TMP are complex, since both deformation and heat influence key microstructural features. The size, shape, and orientation of each grain (crystal) is changed by deformation. New grains may form, consuming the old grains (a process known as *recrystallisation*). Alloying elements can form precipitate particles, either during cooling from elevated temperature deformation, or during a separate precipitation heat treatment. Defects from casting, such as pores and large brittle particles, can be largely eliminated by TMP.

Physical properties of metals and alloys

In some applications, alloys are chosen based on their physical rather than mechanical properties. The key

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physical properties of relevance to metals and alloys are discussed briefly below.

Electrical properties

Most metals are very good conductors of electricity due to their delocalised electrons that respond readily to the application of an electric field. Microstructural features that act as barriers to electron motion decrease the conductivity of metals. Such features include alloying additions dissolved in solid solution and defects such as dislocations and vacancies. The conductivity of alloys also decreases with increasing temperature due to increased thermal vibration of the atoms and an increase in defect density, both of which increase electron scattering.

Thermal properties

The thermal conductivity of metals occurs largely through free electrons, and thermal conductivity is therefore closely linked to electrical conductivity and is influenced by the same microstructural features. Other thermal properties of alloys such as heat capacity and coefficient of thermal expansion (CTE) vary significantly from metal to metal, but are relatively insensitive to microstructural changes for a given alloy composition.

Magnetic properties

Metals may either show a permanent magnetic moment (be magnetic) or show magnetisation only in the presence of an applied electromagnetic field (be non-magnetic). Non-magnetic alloys may be divided into two types: *diamagnetic* and *paramagnetic*. Diamagnetic metals react weakly to an applied field, with the weak induced magnetic moment being in a direction that opposes the applied field. Diamagnetism is so weak that it rarely has any practical consequences. Diamagnetic metals include copper, silver, and zinc. Paramagnetic metals respond to an external field in a way that enhances the field (rather than opposes it, as with diamagnetic metals). This effect, although stronger than that exhibited by diamagnetic metals, is weak compared with the magnetism exhibited by magnetic metals. Aluminium, titanium and chromium are examples of metals that show paramagnetic behaviour.

Magnetism is most important in metals that show a large and permanent magnetisation, which is known as *ferromagnetism*. Iron (in its BCC ferrite form), cobalt and nickel show ferromagnetic behaviour. The strength of the magnetisation depends on temperature. As temperature increases, thermal vibration of atoms results in a decrease in magnetisation until a critical temperature is reached (the Curie temperature), above which there is no permanent magnetisation (and paramagnetic behaviour is shown). Alloying can be used to optimise magnetic properties for a given application. Steels containing chromium and tungsten, Cu-Ni-Fe alloys, and Al-Ni-Co alloys are all magnetic materials that show a high resistance to demagnetisation thanks to favourable microstructures, making them suitable for applications where permanent magnetism is required.

Optical properties

Bulk metals are all opaque throughout the whole visible light spectrum; light radiation falling on a metal is either absorbed or reflected. Reflectivity for most metals is between 0.9 and 0.95 of the incident light energy. Metals that appear bright and silvery do so because they are reflective over the whole light spectrum. Metals that appear coloured (such as copper) do so because there is a bias in the visible light photons they reflect when white light is incident, and photons of certain wavelengths are missing from the reflected spectrum.

Corrosion of metals

In choosing an alloy for a given application, it is critical to consider the environment in which the alloy is to be used. All metals and alloys can be subject to corrosion. In this process, metal interacts with its environment leading to degradation of the metal.

Corrosion of metals is most commonly an electrochemical process that involves both a chemical reaction and a transfer of electrons. Corrosion occurs by the elemental metal undergoing a reaction that results in a loss of electrons, creating metal ions, which are then taken into solution. The chemical driving force for corrosion varies widely from metal to metal and also depends on alloying. Another critical factor is whether the metal can form a protective oxide coating which results in its *passivation* (reduction in chemical reactivity). Aluminium, for example, is a reactive metal but forms a protective oxide film in many environments, giving a high level of corrosion resistance. Stainless steels have a high corrosion resistance under many conditions due to the presence of chromium, which reacts to form a protective surface layer. Care must be taken with metals that rely on passivation for corrosion resistance, since a change in environmental conditions can result in a breakdown of the protective film, and a very large increase in corrosion rates.

Alloying and microstructure both influence corrosion performance. Grain boundaries and second phase particles can act as sites for preferential attack, leading to intergranular and pitting corrosion respectively. The simultaneous action of corrosion and an applied stress can result in greatly accelerated rates of failure, due to stress corrosion cracking.

Particular care has to be taken when metals of different reactivity are used together in a component. Coupling together of two different metals can lead to galvanic corrosion, accelerating the dissolution of the more reactive metal.

This problem can be avoided by ensuring the two metals are not in electrical contact (e.g. by using a non-conductive interlayer).

Alloy selection

Metal alloys are extremely versatile materials, offering a wide range of properties depending on alloy type, composition and microstructure. Steel is by far the most dominant alloy in technological use today, with more than 40 times more steel produced than aluminium, which is the next most widely used metal.

The dominance of steel can largely be attributed to the huge range of different microstructures that can be obtained through changes in composition and processing by exploiting the wide range of solid state phase transformations that are possible. Steel can also be produced with good properties at a cost that can compete effectively with other materials. It should be noted that there are huge ranges of steels of different compositions for different applications. Examples of steel types include plain-carbon steel, high-strength low-alloy steel, stainless steel, tool steel and many others.

Other metals are usually used primarily in applications where there is a special requirement that makes them more suitable than steel. For applications where weight is important such as in aerospace or other transport applications, aluminium or magnesium are often preferred. Aluminium has density that is approximately one-third that of iron, giving aluminium alloys *specific* properties (e.g. specific yield strength = yield strength/density) that can exceed those of steels. Aluminium alloys are also highly formable, and many of them are much more corrosion resistant than plain-carbon steels. Magnesium is lighter still, having a density that is approximately two-thirds that of aluminium. However, as a consequence of their HCP crystal structure, magnesium alloys are more difficult to form. Magnesium alloys can also suffer from high levels of corrosion if not properly protected.

High-temperature applications require alloys that are resistant to creep and thermal fatigue. Nickel-based 'superalloys' are used in aerospace gas turbines (jet engines) to produce components for the hottest part of the engine, where creep resistance is paramount. Since weight is also important in aerospace applications, titanium alloys are widely used where temperatures are lower, since these alloys combine good thermal resistance with lower density than nickel alloys. Titanium alloys are often also highly corrosion resistant and for this reason are used widely in the chemical industry and also in biomedical applications (such as for hip replacements and other implants).

Other alloys have properties that make them suited for applications that are usually dominated by one particular requirement. For example, copper and copper alloys are often used where electrical conductivity is important, and they are also used in architectural applications (e.g. as a roofing material) because of their high corrosion resistance and aesthetic properties (forming a green protective surface film when weathered). Copper is also used with significant additions of other metals to form bronzes and brasses; bronze is most commonly used to refer to alloys based on the copper–tin system, whereas brasses are based on the copper–zinc system. The copper–zinc system in particular can be used to produce a wide range of useful alloys by varying the zinc content, which can be as much as 50% in some brasses. The microstructure of these alloys can either be single phase, with the zinc as substitutional atoms in the FCC copper phase, a two phase mixture of copper-rich FCC and zinc-rich BCC phases, or at the highest zinc levels single BCC phase (containing both copper and zinc). The wide range of possible microstructures explains the versatility of this alloy system.

Other metals that are used as engineering materials include lead and tin alloys. Both have a relatively low melting point and are soft and weak. The advantages of these attributes are that these metals are easy to cast (which requires melting and resolidification) and form. Metals used in significant quantities in special applications include zirconium alloys that are widely used in the nuclear industry for in-reactor components because of their combination of good corrosion performance and low neutron capture characteristics.

Key points

Metal alloys are a versatile class of materials; the addition of the correct alloying elements to a pure metal can lead to large property improvements.

The properties of alloys depend critically on their microstructure, which in turn is strongly dependent on both the alloy composition and processing route.

The deformation of alloys is controlled on an atomic scale by the movement of crystal defects known as dislocations. Alloy strengthening mechanisms work by inhibiting the movement of dislocations.

In addition to mechanical properties, physical properties and corrosion resistance are often key to determining which alloy is best for a given application.

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Chapter 40

Ferrous metals

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Ferrous metals describes steels, cast and wrought iron, and alloy steels in relation to manufacture, composition, physical properties and applications. Paint and metallic coatings for steel are described in relation to corrosion protection. The sustainability of steel as a construction material is discussed.

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Introduction

Steels are defined as the alloys between iron and carbon, but are distinguished from cast iron with its very high carbon content and wrought iron which is virtually free of carbon. In addition to varying the carbon content, the range of steels is extended by the addition of further metallic and non-metallic alloying elements and various heat treatments which modify the mechanical properties and crystal structures. **Figure 1** shows the range of ferrous metals, and the broad effect of carbon content on the key physical properties of steels is illustrated in **Figure 2**.

Steels are categorised by carbon content, but the broad descriptive terminology is applied flexibly to the following ranges of carbon content:

Ultra low carbon steel	<0.01%
Extra low carbon steel	<0.02%
Low carbon steels	0.05–0.15%
Mild steels	0.10–0.25%
Medium carbon steels	0.25–0.5%
High carbon steels	0.5–0.9%
Carbon tool steels	0.9–1.7%

The ultra and extra low carbon steels are used for high formability and drawing applications; the products have a good surface finish. Low carbon and mild steels are relatively soft and can be cold worked due to their ductility and formability; they also can be welded. Most structural steels are within the range 0.13–0.25% carbon content, although pipes and some sections have a low 0.07% carbon content. Cast steel, which is usually normalised, is commonly within the range 0.13–0.25% carbon content, similar to rolled structural steel, the lower carbon contents having greater toughness. Medium carbon steel, which is frequently heat treated, has high strength, is hard-wearing and difficult to weld. High carbon and carbon tool steels exhibit increasing strength, hardenability and wear resistance with increasing carbon content; they cannot be welded and may be brittle.

In addition to the effects of carbon content, steels' properties are significantly affected by the presence of other minor alloying components. Sulfur is always present to some extent, as its total removal would be prohibitively costly, so its maximum content is specified according to the particular application. Current processing techniques can lower the content to 0.01% or less; also the addition of manganese to the steel ameliorates the effects of sulfur by producing manganese sulfide rather than the iron sulfide which causes embrittlement. For machining an increased brittleness caused by higher sulfur content (typically 0.12%) allows faster machining speeds, but sulfur at this level would cause welding failure. The addition of up to 0.15% phosphorus is used to harden steel, but again greater quantities cause embrittlement. As with sulfur, phosphorus is always present to some degree, and its content is also controlled according to the anticipated application, particularly where welding is anticipated. Nickel, chromium and niobium, as minor alloying components, enhance strength, and molybdenum can improve impact resistance.

Typical applications for the various carbon content steels are listed in **Table 1**. **Figure 3** illustrates structural steelwork in the construction of Wembley Stadium.

Cast iron

Cast iron is an alloy of iron with a carbon content within the range 2–4% and typically 1–3% silicon. It is commonly manufactured from blastfurnace pig iron, scrap and coke in a cupola furnace, and refined to the appropriate carbon content. Alternative production methods involve processing in a coreless induction furnace or an electric arc furnace. The low melting point (approximately 1150°C) and the high fluidity when molten give rise to its excellent casting properties, but it cannot be hot worked, and it is generally a brittle material. Different grades of cast iron are associated with distinct microscopic crystal structures. The designation systems for cast iron based on symbols and on numbers are described in BS EN 1560: 1997, and examples of the graphitic microstructures are illustrated in BS EN ISO 945: 1994.

Types of cast iron

Grey cast iron

Grey cast iron is the common form with 2.5–4% carbon and 2% silicon. Much of the carbon exists in the form of

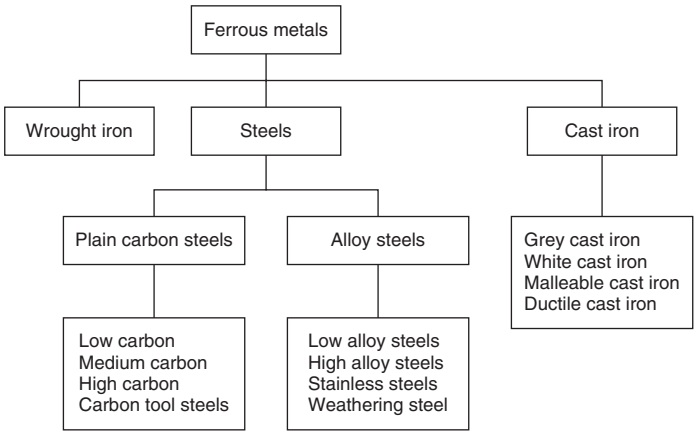

Figure 1 The range of ferrous metals

graphite flakes within a ferrite crystal matrix. The graphite is visible when the brittle material is fractured. BS EN 1561: 1997 describes, for a range of grey cast irons, the tensile properties which can vary with cooling rate and section thickness. Thin sections can have reasonable tensile strengths. This is associated with some of the carbon remaining as cementite crystals (Fe_3C), within a pearlite matrix producing a stronger and harder material. Grey cast iron expands on solidification to give sharp castings. Although brittle and with poor shock resistance, grey cast iron can be machined. It has good compressive strength and corrosion resistance but is very difficult to weld. It is used for machine blocks where it is effective at damping vibrations and historically for structural units in compression. For locations where impact resistance is not critical, certain road iron goods (BS EN 124: 1994) such as gully tops, manhole covers and pipes, are manufactured from grey cast iron.

White cast iron

White cast iron has a structure of pearlite within a cementite matrix, and is very hard, brittle and difficult to machine. It can however be annealed to malleable cast iron (BS EN 1562: 1997). The greater hardness of white compared with grey cast iron, together with its wear and abrasion resistance, makes it appropriate for use in machinery and moulds.

Malleable cast iron

Different grades of malleable cast iron are manufactured from white cast iron by annealing through different time,

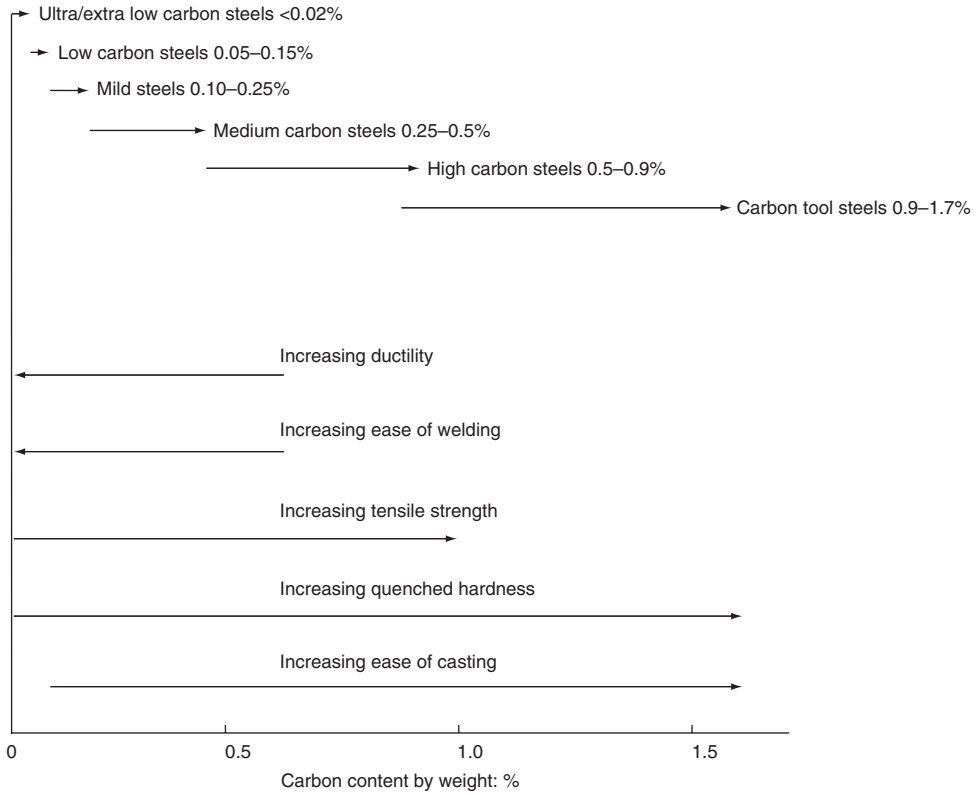

Figure 2 The effect of carbon content on the properties of steels (adapted from A. Lyons, Materials for Architects and Builders, 3rd edition ©2007 Elsevier)

	Ultra low carbon steel	Extra low carbon steel	Low carbon steel	Mild steel	Medium carbon steel	High carbon steel	Carbon tool steel
Carbon content: %	<0.01	<0.02	0.05–0.15	0.10–0.25	0.25–0.5	0.5–0.9	0.9–1.7
Alternative treatments	Normalised, may be work hardened	Normalised	Normalised	Normalised	Quenched and tempered	Quenched and tempered	Quenched and tempered
Properties	Ductile High formability Weldable	Ductile Good formability Weldable	High strength Ductile Weldable	High strength Ductile Weldable	High strength High toughness Difficult to weld Brittle below room temperature	High strength Low ductility Difficult to weld	Very hard Low toughness Wear resistant Cannot be welded May be brittle
Applications	Automotive sheets, wire mesh	Automotive sheets, pipes	Offshore engineering, pipes	Structural rolled sections, tubes, plates, pipes, cast steel	Machinery rails	Machinery	Machine tools dies, springs

Table 1 Typical properties and applications of steels according to carbon content

temperature and atmosphere cycles. The three principal grades of malleable cast iron are whiteheart, blackheart and pearlitic. Whiteheart is produced by annealing in a decarburising atmosphere, creating a difference in composition between the surface material which is pure ferrite and the core which is composed of pearlite, ferrite and temper carbon (nodules of graphite). Blackheart is produced by slow annealing at a high temperature (900–950°C for several days) producing a matrix of ferrite with temper carbon nodules. However, the material can be produced more rapidly by small additions of magnesium or cerium on casting. Pearlitic malleable cast iron has a matrix of pearlite or tempered martensite with carbon nodules, produced by high-temperature annealing followed by more rapid cooling in air or quenching in oil giving a higher strength. Malleable cast iron has greater tensile strength and impact resistance than grey cast iron. It bends before breaking and can be welded. It is used in machinery including tools and gears.

Figure 3 Structural steelwork in the construction of Wembley Stadium (Architects: Foster and Partners. Photograph by Arthur Lyons)

Ductile cast iron

Nodular ferritic ductile iron (spheroidal graphite cast iron) (BS EN 1563: 1997) is manufactured by the addition of small quantities of magnesium to the melt before casting. The material consists of nodules of graphite within a matrix of ferrite and/or pearlite. The material has enhanced properties of improved strength, ductility and hot workability compared to grey cast iron. Heat treatment of ductile cast iron produces the higher-strength austempered ductile cast iron (BS EN 1564: 1997). Ductile cast iron has good casting properties, impact and fatigue resistance, and can be welded. It is used in machinery applications, water and sewerage pipes (BS EN 545: 2006 and BS EN 598: 2007 respectively) and for road iron goods where impact resistance is important.

Wrought iron

Production of 'puddled' wrought iron ceased in the UK in 1974, so any new wrought iron is either recycled or, more frequently, low carbon steel. Traditional wrought iron contains approximately 0.02% carbon. Because of its manufacturing process, wrought iron is fibrous in character, incorporating up to 5% veins of siliceous slag residues. Wrought iron has a melting point of around 1540°C depending upon its purity. It is ductile and easily forged when hot (1350–1450°C) and has a greater resistance to corrosion than low carbon steel. Traditionally, in addition to the manufacture of ornamental ironwork, it was used for tension components such as bolts and chains, due to its tensile strength of approximately 350 MPa. Recycled 'puddled' wrought iron is commercially available in certain stock sizes, particularly for specialist conservation work, where its replacement with mild steel would be inappropriate.

Steel

Manufacture

The production of steel is a multi-stage process, involving the production of impure iron in a blastfurnace, followed by its conversion into steel by either the basic oxygen or electric arc process. Molten steel is then cast and subsequently hot-rolled. Thin sections may then be cold-rolled and coated as required.

Manufacture of iron

Iron ore, which is essentially naturally occurring iron oxide, is blended to give an appropriate composition then heated with coke to form a granular sinter. The sinter, coke, iron ore and limestone are fed into the top of the blastfurnace. Hot air is blown into the base of the furnace through the tuyères, raising the temperature to 2200°C to melt the raw materials and cause the chemical reduction of the iron oxide to metallic iron. The limestone fuses to produce a slag on the surface which absorbs some impurities from the molten iron. Molten metal collects at the base of the furnace and is tapped into a ladle when required for conversion into steel. The slag is run off for use as a by-product in cement manufacture and aggregate in the construction industry. The iron produced contains impurities of sulfur, phosphorus, manganese and silicon which are removed during the subsequent conversion to steel which also reduces the carbon content from approximately 4%. Iron production is continuous, with a blastfurnace shutting down only for refractory brick relining after several years of operation. A large blastfurnace working at maximum capacity can produce up to 10 000 t of iron per day.

$$\underset{\text{carbon (coke)}}{2C} + \underset{\text{oxygen}}{O_2} \rightarrow \underset{\text{carbon monoxide}}{2CO}$$

$$\underset{\substack{\text{iron ore}\\ \text{(hematite/magnetite)}}}{Fe_2O_3} + \underset{\text{carbon monoxide}}{3CO} \rightarrow \underset{\text{iron}}{2Fe} + \underset{\text{carbon dioxide}}{3CO_2}$$

Basic oxygen process

The basic oxygen process is used for the bulk production of steel. A refractory lined steel furnace is charged with scrap metal, followed by molten iron directly from the blastfurnace. A water-cooled lance blows high-pressure pure oxygen just above the surface of the metal. The oxidation of the impurities produces heat and refines the metal. The temperature is controlled by the addition of iron ore and scrap metal; lime is added to produce a slag. The refining is continuously monitored and adjusted by blowing argon, nitrogen or oxygen through the base of the converter. The carbon content is reduced from 4% in the 'hot metal' from the blastfurnace to approximately 0.05%, and other alloying components are added as appropriate when the molten steel is tapped. A typical basic oxygen furnace will convert 350 t of iron into steel within 30 minutes.

Electric arc process

The electric arc process is used mainly for recycling scrap steel. The furnace consists of a refractory lined hearth with a removable lid through which graphite electrodes, which heat the furnace, can be raised and lowered. The furnace is charged with scrap steel and appropriate quantities of alloy steel to produce the required composition. The powerful electric arc melts the scrap and the molten steel is purified by a lime slag and the oxygen atmosphere. The process uses approximately half the energy per tonne of recycled steel compared with manufacturing new material by the basic oxygen process. A typical furnace will produce 150 t of steel or stainless steel within 90 min.

Continuous casting

In order to save energy, most steel is continuously cast directly from the steel-making process into slabs, blooms or billets. Molten steel is transferred from the basic oxygen or electric arc furnace in a teeming ladle and run into a 'tundish'. This supplies molten metal at a controlled rate into a series of refractory tubes, then water-cooled copper moulds to create a continuous block of solid steel which is automatically cut to length by gas burners as billets, blooms or slabs depending on its shape, for transfer to the rolling mills. Flat slabs are used for rolling into plate and sheet, blooms for structural sections and billets for smaller sections including rod.

Continuous cast steel is normally 'killed' by the addition of aluminium and/or silicon at casting to remove all oxygen and prevent the evolution of gases, particularly carbon monoxide, during solidification.

Although most steel for construction is produced as rolled plate, sheet or section, a small proportion is directly cast. Units such as nodes for circular hollow section structural systems may be made as individual castings. They can be subsequently welded in situ to the standard milled steel hollow sections to give structural continuity.

Forming steel

Hot-rolled steel – heavy sections, plate and coil

The majority of steel forms are produced by the repeated hot-rolling of billets, blooms or slabs at approximately 1250°C through a series of 'mill stands' until the required shape and dimensions are achieved. This process is used to produce the standard construction sections including columns, beams, channels and angles in addition to flat plate and coiled sheet. High-strength hot-rolled low-alloy steel with good welding properties is used in building including bridge construction. Hot-rolled steel floor plate is manufactured with a self-draining non-slip patterned

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surface. Hot-rolled sheet and strip with good formability to BS EN 10111: 2008 is subsequently used for cold forming by bending and deep drawing, leading to the manufacture of pipes and industrial furniture. Hot-rolled high carbon steel is used in the manufacture of quenched and tempered machine parts.

Cold-rolled steel – light sections and profile sheet

Cold rolling is used to further reduce the thickness of sheet steel. A higher quality of finish is produced by cold rolling and the cold working increases the tensile strength of the steel. High strength grades may be annealed or full hard condition. Coiled steel may be rolled into profiled sections for roofing and cladding sheets or into light structural sections for internal stud partitions. BS EN 10209: 1996 details cold-rolled low carbon steel suitable for vitreous enamelled architectural wall panels. Cold-rolled high-strength steel (BS EN 10268: 2006) alloyed with aluminium, and some titanium or niobium, is appropriate for use in internal structural components and tubes. The standard BS 1449-1.1: 1991 details the broad range of carbon and carbon-manganese steels that may be cold rolled for structural steelwork and tubing furniture. The higher quality finish of cold-rolled steel is suitable for the application of zinc, tin or paint coatings.

Forming hollow sections

Welded hollow steel sections are manufactured by bending flat sheet into a tube which is then continuously welded either by high-frequency electric resistance induction welding or by arc welding. Tubes for construction are smoothed off, normalised by heat treatment to unify their strength properties, then stretched, reduced or rolled into the required circular, oval, square or rectangular structural hollow sections.

The rotary forge process is used for the production of seamless hollow tubes, particularly heavier sections. A hot cylindrical or tapered ingot is pierced by a hydraulic ram; the central void is then opened up by the action of rollers and a rotating mandrel. Subsequently the steel passes through a series of eccentric rollers which elongate the tube reducing the section thickness to the required dimension.

Bending curved structural sections

Specialist manufacturers use progressive roller bending to produce curvature on mild steel and stainless steel standard sections, castellated beams, solid and hollow sections and tapered beams, without visible distortion, as illustrated in **Figure 4**. Roller bending to coordinates with variable or reversed curvatures and bending in three dimensions are all possible. Generally smaller sections can be bent to tighter radii than larger sections; however, for a given section size the thicker gauge units can be bent to tighter radii than the thinner gauge products. Press braking is used for bending sheet steel along its full length in one pressing.

Figure 4 Curved hollow section steelwork at Renaissance Bridge, Bedford (courtesy of The Angle Ring Company Ltd)

Drawing wire

Steel wire is manufactured by drawing annealed rod through a series of lubricated tungsten carbide tapered dies to produce the required elongation and reduction in diameter. The process reduces ductility and increases the strength of the steel. Special heat treatment is therefore required to increase the ductility of high-tensile steel sufficiently to facilitate the drawing process. Steel cables for prestressed concrete or suspension/tensile structures are manufactured by twisting wires into strand. A series of strands is woven around a central core strand to produce rope or cable of the required dimensions and load-bearing capacity. Typical configurations use 7 or 19 strands or ropes to form heavy-duty cables, frequently in grade 1.4401 stainless steel.

Casting components

Although most steel for construction is produced as rolled plate, sheet or section, a small proportion is directly cast. Units can subsequently be welded in situ to the standard milled steel sections to give structural continuity. The standard BS ISO 19960: 2005 refers to cast steels with special physical properties. Castings include nodes for hollow section structural systems and special components for fixing tension members where architectural requirements preclude the use of standard sections.

Manufacturing meshes and nets

Perforated sheets with a range of open area are manufactured in both mild and stainless steel. Stainless steel units

usually have polished or brushed finish, whilst for exposed applications mild steel is galvanised or powder coated. Very fine holes, either round or square, to micron scale can be laser cut. Both expanded metal and woven meshes are appropriate for architectural screens and other applications.

Fabrication

Fabrication processes include circular saw cutting for accurate section lengths. Plasma arc cutting is used for more complex cuts. An electric arc is struck between the copper nozzle and the steel, and a fast gas plasma flow from the nozzle removes the arc-fused dross at 20 000°C during the cut. The plasma gas may be air, oxygen or an inert gas depending on the particular system. Oxyacytylene cutting burns the steel to iron oxide at a temperature of 700–900°C. Holes in plates up to 15 mm may be punched, but for greater plate thicknesses drilling is required. Metal active gas welding with an arc electrode and an argon/carbon dioxide shield is appropriate for small sections, whereas submerged arc welding with powder protection of the welded joint is necessary for large steel sections.

Steel is frequently shot-blasted to produce a good 70 micron textured finish for subsequent welding or painting. Most structural steelwork is fabricated offsite for immediate bolt fixing, reducing construction time on site.

Composition of steel

The extensive range of steels arises from varying compositions and heat treatments. Each product, including technical delivery conditions, may be specified either by the alpha-numeric steel name (BS EN 10027-1: 2005) or by the unique steel number (BS EN 10027-2: 1992). The following examples for a standard structural steel and a weathering steel illustrate the operation of the Steel Name and Steel Number coding systems.

Steel name

The principal symbols are the letter S (structural steel) or G (steel casting) and the numbers that refer to minimum yield strength (MPa) for the smallest thickness range.

The additional symbols refer to impact property J (low impact energy) and K (high impact energy) at the specified temperatures R, 0, 2, 3, 4, 5 and 6 = 20, 0, −20, −30, −40, −50 and −60°C respectively.

Further additional symbols, where required, refer to treatments, end uses or special chemical composition (e.g. N – normalised, W – weather resistant, H – hollow section, M – thermomechanically rolled, Q – quenched and tempered, T – tubes, and D – hot dip coating, F – forgings).

S275J0
Structural steel with a yield strength is 275 MPa at 16 mm.
J is the lower impact energy and at 0°C.
(Longitudinal Charpy V-notch impacts 27 J @ 0°C.)

S355K2W
Structural steel with a yield strength is 355 MPa at 16 mm.
K is the higher impact energy and at −20°C.
W refers to weather-resistant steel.

Steel number

The first digit is the material group number with steel 1.

The first decimal pair of digits is the steel group number with, for example, 01 referring to a non-alloy quality steel. Special alloy steels have numbers between 20 and 89, for example, while stainless steels have numbers between 40 and 46.

The final two digits refer to the particular grade of steel.

1.0143
1. Steel.
01 General structural non-alloy steel – tensile strength <500 MPa.
43 Sequential number refers specifically to the particular steel.

1.8966
1. Steel.
89 High-strength weldable alloy steel not intended for heat treatment by the user.
66 Sequential number refers specifically to the particular steel.

The standard BS EN 10020: 2000 gives a definition of classes of steel as non-alloy steels, stainless steels and other alloy steels, by chemical composition. The standard BS EN 10021: 2006 defines the delivery conditions for steel products and BS EN 10079: 2007 defines the range of steel products including a list of references to further European standards on each product type (e.g. tubes, sections, prestressing wire, stainless steel, etc.).

Grades of steel

Structural steels

The following series of European Standards relate to structural steels.

BS EN 10025-1: 2004
General technical data.
BS EN 10025-2: 2004
Non-alloy structural steels.
BS EN 10025-3: 2004
Weldable fine-grained structural steels.
BS EN 10025-4: 2004
Rolled weldable fine-grained structural steels.
BS EN 10025-5: 2004
Steels with improved atmospheric corrosion resistance.
BS EN 10025-6: 2004
High yield strength structural steels.
BS EN 10210-1: 2006
Hot finished structural hollow sections.

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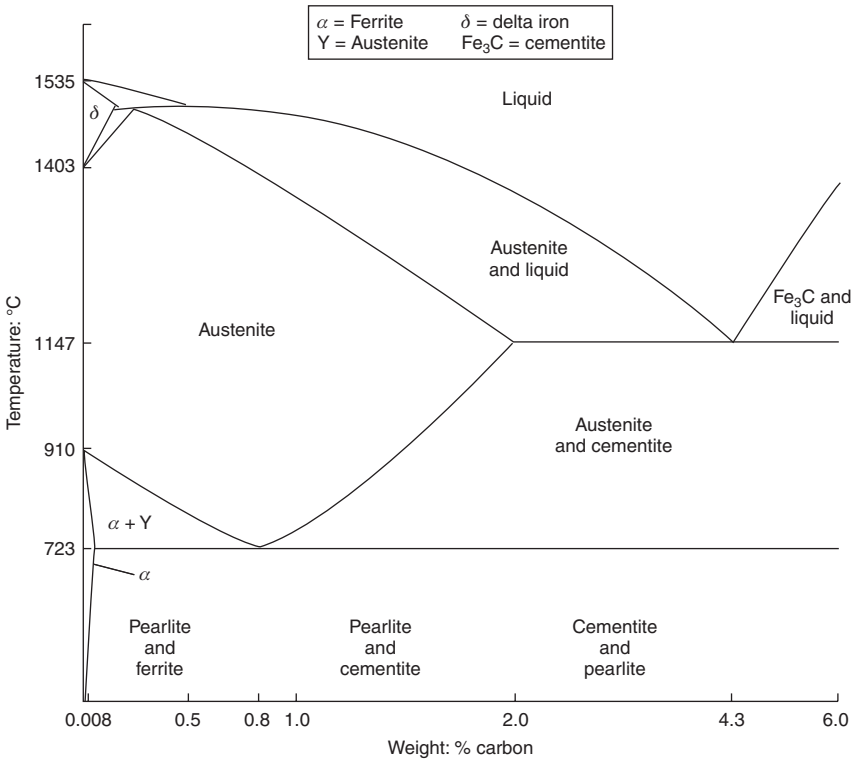

Figure 5 The iron–carbon phase diagram

BS EN 10219-1: 2006

Cold-formed welded structural hollow sections.

The standards list a range of steels by grade, steel number and key physical properties.

Microstructure of steel

The phase diagram, in **Figure 5**, for iron and carbon, shows the forms of the iron and carbon alloy depending upon composition and temperature. Ferrite (α-iron) essentially pure iron with low carbon solubility exists up to 910°C. Above this temperature the solid solution of carbon in iron (up to 2% at 1147°C) is austenite (γ-iron). Cementite is a hard and brittle compound of 6.7% carbon in iron with a chemical composition of Fe_3C. Above 1403°C iron exists as δ-ferrite.

The rate of cooling from the high-forming temperature determines the proportion of the high-temperature crystal structure remaining at room temperature. Thus if austenite with less than 0.8% carbon is cooled slowly it converts to pearlite consisting of alternate plates of cementite and ferrite in a matrix of ferrite. An austenite with greater than 0.8% carbon will similarly convert to pearlite but within a matrix of cementite.

If, however, austenite is cooled rapidly, preventing migration of the carbon atoms, then martensite, which is a distorted ferrite containing trapped carbon, is formed. This material is strong and relatively brittle. With an intermediate cooling rate, bainite which is between pearlite and martensite in character is produced. This form is harder, stronger and tougher than pearlite.

Heat treatments

The range of heat treatments to modify the physical properties of steels is extensive, but the standard treatments are hardening, normalising, annealing, tempering and carburising. The full range of heat treatments is listed in BS EN 10052: 1994.

Hardening

Steels in the range 0.3–0.6% carbon are frequently hardened by heating to a high temperature followed by quenching in oil or water. If the material is excessively hard and brittle due to the retention of the high-temperature crystal structure, it is subsequently tempered to the required physical properties.

Normalising

Normalising involves heating to 830–950°C, depending upon carbon content, followed by cooling in air. The process relieves stresses and softens the material to permit cold working and machining.

Annealing

Annealing requires soaking at a high temperature, typically 500–650°C, depending upon the carbon content, for a predetermined period followed by slow controlled cooling giving an equilibrium microstructure. The process generally produces the softest steel for a given composition.

Tempering

Tempering is the reheating of steel to a moderate temperature (400–600°C) to allow some recrystallisation of the higher temperature brittle structure produced by rapid quenching. The extent of the tempering is related directly to the temperature and timescale of the process. Ductility increases and strength decreases with increased tempering temperatures. The Standard BS EN 10083 Parts 1, 2 and 3: 2006 detail the technical delivery conditions for steels for quenching and tempering.

Carburising

Carbon or alloy steels (usually less than 0.25% carbon) may be case hardened to produce a hard-wearing higher carbon content surface material over the unmodified softer core material. The carburising process usually involves soaking the manufactured components at 900°C for several hours in a furnace whilst surrounded by carbon.

Steel in reinforced concrete

The specifications of steels for plain, ribbed and indented wire for the reinforcement of concrete, steel fabric and weldable reinforcing steel are described in the standards BS 4482: 2005, BS 4483: 2005 and BS 4449: 2005 respectively, which are used in conjunction with BS EN 10080: 2005. The minimum tensile properties and appropriate chemical compositions are noted. The type, dimensions and physical properties of fibres for concrete are described in the standard BS EN 14889-1: 2006. Terminology is described in BS ISO 16020: 2005.

Tension components

Steels for tension components, including rope end connectors, anchorages and their corrosion protection, are described in Eurocode 3 – BS EN 1993-1–11: 2006.

Alloy steels

A wide range of alloy steels is produced for specific end uses. However, apart from stainless and weathering steel, carbon-manganese steels and micro-alloy steels are important within construction. Carbon-manganese steels are typically 1.2% carbon steels with 1.8% manganese added to improve strength and toughness.

The high-strength low-alloy (HSLA) or micro-alloyed steels have good weldability, similar to that of mild steel, and with high strengths in rolled or noramalised condition. They are manufactured by adding between 0.05 and 0.10% of additional alloying elements to carbon-manganese steel with a low sulfur and phosphorus content (typically 1.6% manganese). Additions include boron, vanadium, titanium and niobium, which control grain size producing a material which may be forged without subsequent costly heat treatment. Other additions may include molybdenum, silicon and zirconium. Micro-alloyed steels also have good toughness and fatigue strength due to their refined microstructure.

Stainless steel

Stainless steels form a group of ferrous alloys containing a minimum of 10.5% chromium and a maximum carbon content of 1.2%. The standard BS EN 10088-1: 2005 gives an extensive list of the compositions and properties with the relevant steel names and steel numbers. BS EN 10088 Parts 2 and 3: 2005 describe the approved technical delivery conditions for various production forms. The materials commonly used in construction are categorised by metallurgical terminology into austenitic, ferritic and austenitic/ferritic (duplex) stainless steels. The martensitic stainless steels are not normally used in construction. **Table 2** lists the standard materials and suitable working environments.

Stainless steel is manufactured by a two-stage process. Initially scrap metal is melted in an electric arc furnace. The hot metal is then transferred to an argon–oxygen decarburiser where it is refined. On tapping, appropriate alloying additions are made prior to transfer of the molten metal to a continuous casting machine for forming and subsequent fabrication.

Designation/type	Name (indicating composition of alloying components)	Number	Suitable environments
Austenitic	X5CrNi18-10	1.4301	Rural and clean urban
	X5CrNiMo17-12-2	1.4401	Urban, industrial and marine
Ferritic	X6Cr17	1.4016	Interior
Austenitic/ferritic (duplex)	X2CrNiMoN22-5-3	1.4462	Severe industrial and marine

Notes: Cr, Ni, Mo and N refer to chromium, nickel, molybdenum and nitrogen respectively. X2, X5 and X6 refer to the carbon contents of 0.02, 0.05 and 0.06% respectively. This table includes data derived from British Standards. British Standards can be obtained in PDF format from the BSI online shop: http://www.bsi-global.com/en/Shop/ or by contacting BSI Customer Services for hard copies: Tel: +44 (0)20 8996 9001; email: mailto:cservices@bsi-global.com

Table 2 Stainless steel compositions and grades to BS EN 10088-1:2005 for different environmental conditions

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The corrosion resistance of stainless steel is due to the passive film of chromium oxide which forms over the surface of the metal on exposure to atmospheric oxygen. The film is self-healing if the surface is scratched or damaged. The corrosion resistance is increased by the inclusion of nickel and molybdenum, therefore the ferritic alloys (e.g. 1.4016) are only appropriate for interior use where corrosion is not a critical factor. The austenitic alloys (e.g. 1.4301 and 1.4401) are the standard products used for exterior roofing, cladding and fitments. The duplex alloys (e.g. 1.4462) are used for applications within severe industrial or marine environments.

In highly aggressive environments, stainless steel can be susceptible to localised corrosion by pitting, crevice corrosion or bimetallic corrosion. Pitting or crevice corrosion may occur in chloride-contaminated environments where either the natural oxide film is penetrated or where differential oxygen levels between the crevice and exposed regions exist, typically at fixings. Bimetallic corrosion is liable to occur where dissimilar metals are in electrical contact in the presence of moisture. For example, carbon steel bolts should be avoided in fixing stainless steel, as the bolts will be subject to corrosion. Dissimilar metals should be electrically insulated and contact with copper should be avoided. The standard BS EN 1993-1-4: 2006 lists the range of alloys appropriate for use under various atmospheric conditions and offers design advice on the avoidance of potential corrosion risks.

Stainless steel finishes include polished, matt, brushed, patterned and profiled. Additionally coloured finishes can be produced by the incorporation of dyes into the surface oxide film by chemical and cathodic processes, giving a range of colours from bronze, gold and red to purple, blue or green according to the final thickness.

Stainless steel is available in sheet form, bars, standard and hollow sections. It is widely used for cladding, roofing and trim owing to its combined strength, low maintenance and aesthetic properties, as illustrated by the Lowry Centre in Manchester (**Figure 6**). The corrosion resistance of stainless steel makes it appropriate for use in fixings, such as tensile wires and anchors, wall ties, corbel plates and concrete reinforcement. In addition it is widely used in plumbing and catering facilities where a quality finish is required.

Weathering steel

Weathering steels (BS 7668: 2004) are ferrous alloys containing small proportions of copper, usually between 0.25 and 0.55%, together with other alloying components, typically chromium, silicon, manganese, phosphorus and minor quantities of aluminium, titanium, vanadium or niobium. The alloying ensures that the brown iron oxide rust adheres to the surface, preventing further corrosion after the initial exposure. Weathering steel should not be used in marine environments and all exposed steel should be carefully detailed to ensure that initial rainwater run-off does not cause staining to other adjacent materials, as the brown coloration develops over the first years of exposure. Grades of weather-resistant steels are listed in the standard BS EN 10025-5: 2004. Weathering steels are used in structural (**Figure 7**) and cladding applications and also for townscape sculptural works of art.

Figure 6 Stainless steel façade of the Lowry Centre, Manchester (Architects: James Stirling and Michael Wilford) (courtesy of Corus)

Coatings

Coating systems are required on exposed steelwork to form a barrier between the steel and its environment to prevent corrosion. This involves the preparation of the metal surface followed by the application of an appropriate metallic or non-metallic coating. Cleaning and surface preparation techniques are listed in standards BS 7773: 1995 and BS EN ISO 12944-4: 1998.

Figure 7 Structural weathering steel, Motorway M4 at Chieveley (courtesy of Corus)

The first element in the cleaning cycle is degreasing using organic solvents, emulsion cleaners or alkali washing. This is followed by grit or shot-blast cleaning to remove any rust or mill scale and to produce a 70 micron textured finish which is good for welding and promotes good adhesion for paint finishes.

Paint systems

Paints consist of a blend of components, each having a specific function. Usually these include the binder which solidifies to produce the paint film, a solvent to ensure fluidity during application, a base, often titanium dioxide, to give opacity and pigments to give colour. Other additives include inert extenders and also driers to accelerate the setting or polymerisation of the binder.

Paints are usually applied as a three-coat system for maximum corrosion protection. The primers are normally zinc-rich compounds in alkyd or epoxy-ester resins which adhere well to the cleaned surface, form a good base for subsequent coats and provide corrosion inhibition. Alternatively, acrylated rubber paint may be used as a primer to form a physical barrier over steelwork. In severe environments two-part epoxy or polyurethane paints mixed immediately prior to application are advisable. Undercoats increase the overall film thickness, whilst providing cover and a good base for the finishing coat which gives the colour and weather protecting gloss or satin finish. Details of a broader range of protective paint systems are given in the standard BS EN ISO 12944-5: 1998, following the general introduction in Part 1: 1998 and the classification of environments in Part 2: 1998.

Metallic coatings

Cold-rolled steel suitably cleaned has a good surface for subsequent coating with zinc, zinc alloys or terne.

Zinc and zinc alloys

Zinc finishes to steel are typically zinc (Z), zinc–iron alloy (ZF) and zinc–aluminium alloy (ZA). A zinc coating may be applied by hot-dipping in 99% molten zinc at approximately 455°C or by spraying with molten zinc. Sheradising in heated zinc powder or electrodepositioning are used for coating small components. Hot-dipped zinc-coated steel may have a normal (N) spangled finish associated the crystallisation of the zinc, or a minimised (M) spangle finish if the crystalline appearance would adversely affect subsequent painting or coating.

Zinc–iron alloy (ZF) coating is produced by the heat treatment of hot-dipped zinc-coated steel. The annealing process allows some iron to diffuse into the zinc layer producing a coating of 8–12% iron. The alloy surface has a uniform matt grey appearance.

Zinc–aluminium alloy (ZA) coating is produced by hot-dipping the steel in a 5% aluminium in zinc alloy. The alloy has a metallic lustre, which varies with the production conditions which control the crystallisation of the metal coating.

The durability of galvanised steel is directly related to the thickness of the coating and the environmental conditions. The standard coating mass for zinc in hot-dipped sheet steel is 275 g/m^2 (i.e. 137.5 g/m^2 on each face), but this can be increased for example to 350 g/m^2 for greater durability. Standards BS EN 10326: 2004 and 10327: 2004 specify the requirements for hot-dip coated structural steel and sheet steel respectively. **Table 3** illustrates the corrosion risk categories associated with different environments listed in BS EN ISO 14713: 1999.

Aluminium alloys

Aluminium zinc (AZ) coated steel is produced by hot-dipping in a molten bath composed of 55% aluminium,

Corrosion		Typical environment	Typical corrosion rate in microns (μm) per year	
Category	Risk		Low-carbon steel	Zinc
C1	Very low	Interior – dry	<1.3	<0.1
C2	Low	Interior – occasional condensation Exterior – exposed rural inland	1.3–25	0.1–0.7
C3	Medium	Interior – high humidity, some air pollution Exterior – moderate urban or mild marine	25–50	0.7–2.1
C4	High	Interior – swimming pools, chemical plants Exterior – industrial inland or urban coastal	50–80	2–4
C5	Very high	Interior – aggressive industrial with high humidity Exterior – aggressive industrial or marine coastal	80–200	4–8

Note: Environmental conditions change with time, hence these values should only be used as indicative.

This table includes data derived from British Standards. British Standards can be obtained in PDF format from the BSI online shop: http://www.bsi-global.com/en/Shop/ or by contacting BSI Customer Services for hardcopies: Tel: +44 (0)20 8996 9001, Email: mailto:cservices@bsi-global.com

Table 3 Corrosions risk associated with different environments for sheet low-carbon steel and zinc sheet (BS EN ISO 12944-2: 1998 and BS EN ISO 14713: 1999) (reproduced with kind permission from BSI)

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1.6% silicon and 43.4% zinc. The normal coating has a metallic spangled finish associated with the crystal growth during solidification.

Hot-dipping in a molten alloy of aluminium with 8–11% silicon (AS) produces an alloy finish to the steel composed of aluminium, iron and silicon.

Generally, aluminium coatings are more durable than the equivalent zinc coatings and may be used without further protection in non-aggressive environments. They are also used as the substrate for certain organic coatings.

Terne coating

Terne is an alloy of lead (80–90%) and usually tin (20–10%). Is may be applied as a hot-dipped coating to steel or stainless steel, particularly for use in cladding and roofing. The coating is designated by mass per square metre. For example, a coating with a mass of 170 gm/m^2 (total both sides) has a thickness of approximately 0.020 mm. Terne-coated stainless steel does not suffer from bi-metallic corrosion and can normally be used in contact with lead, copper, aluminium or zinc. Thermal movements are similar to the base stainless steel, and units up to 9 m may be used for roofing or cladding. Details of the coating designations are listed in the standard BS ISO 4999: 2005.

Lead clad steel

Lead clad steel is produced by cold-roll bonding typically 0.75 mm lead to 1.0 mm terne-coated steel or stainless steel. The material has the appearance and corrosion resistance of milled lead but with the self-supporting properties of steel. Patination oil may be applied as in traditional lead systems. The material is used as a roofing and cladding material, and for small units such as fascias, soffits and gutters. Joints can be lead burned or soldered as traditional lead. Exposed steel, although not stainless steel, should be protected by soldering.

Non-metallic coatings

Steels for vitreous enamelling are described in the standard BS EN 10209: 1996. The range of plastic coatings on steel includes materials applied both as monochromatic liquids and printed plastic films. The most widespread material is the PVC plastisol, which is used extensively for cladding in commercial and industrial buildings. The coated steels meet the Class 0 fire performance requirements for the UK Building Regulations – 2000. Generally, the period to first maintenance is less for roofs than walls and is also reduced by marine environments or within hotter southern European climates.

PVC plastisol

In the manufacture of PVC plastisol coated sheet, the steel strip is annealed in a hydrogen-rich environment at 700°C, followed by hot-dip coating with zinc/aluminium alloy to a smooth matt surface. The coils are degreased followed by the application of the prime and finish coats and curing at 250°C to polymerise the PVC. The 0.2 mm tough leather grain finish is available in a wide range of colours, but pastel shades are recommended for roofing applications. The reverse side is usually coated with a grey corrosion-resistant primer and a polyester finish, although thick finishes may be specified where necessary. On-site care is required to prevent damage to the surface finish, which should give up to 30 years to first maintenance for roofs and walls except for deep colours and marine environments. A 0.12 mm finish of metallic appearance is also available with a 20-year period to first maintenance for use in walls within temperate northern European environments.

Polyvinylidene fluoride (PVDF)

Polyvinylidene fluoride is applied as a 0.027 mm coating to zinc-coated steel. The finish is smooth and self-cleaning with good colour stability. As the coating is thin, the material requires careful on-site handling to give a 15-years period to first maintenance for walls in non-coastal locations.

Polyurethane

Polyurethane is applied as a 0.05 mm coating to zinc or zinc–aluminium alloy coated steel. It is available in a range of solid and metallic colours with a 25-year period to first maintenance on zinc–aluminium alloy coated steel for walls, or as a textured finish on zinc-coated steel for walls with a 20-year period to first maintenance in non-marine environments.

Polyester

Polyester coatings on galvanised steel offer an economical, less durable 0.025 mm smooth finish with a medium-term life in non-aggressive drier climates and typically a 10-year period to first maintenance. Polyester coatings are also manufactured specifically for internal partitioning, etc.

Polymer film

Polymer films, mainly in PVC, offer a range of smooth and embossed, coloured and patterned, including woodgrain, finishes for internal environments, particularly office interiors.

Enamel coating

Organic enamel-coated steels offer good light reflectance for internal roof and wall linings. Coatings, usually 0.022 mm thick, are usually applied to zinc–aluminium alloy coated steel and are easily cleaned. A range of light colours including white is available.

Corrosion

Corrosion is an electrochemical process, which can only occur in the presence of an electrolyte and oxygen. The overall effect is summarised by the chemical equation:

$$\underset{\text{iron}}{4Fe} + \underset{\text{oxygen}}{3O_2} + \underset{\text{water}}{2H_2O} = \underset{\text{rust}}{2Fe_2O_3 \cdot H_2O}$$

The following are factors that affect the rate of corrosion:

- the total time at which water is present, particularly in crevices and joints
- the presence of pollutants, particularly sulfates and chlorides within the moisture
- the presence of dissimilar metals in mutual contact
- surface effects associated with cold working or welding
- variability of oxygen access into crevices and joints.

Prevention of corrosion in structural steelwork

In order to minimise corrosion, cavities and crevices should be avoided as far as possible, so welded joints are preferable to bolted joints. Standing water should be reduced by the provision of drainage holes. Appropriate and effective coatings should be applied after surface preparation involving the removal of grease, mill scale and rust. Access for maintenance must be considered at the design stage.

Sustainability

Primary steel production requires a large energy input associated with the mining and transportation of the ore mainly from the Americas, Australia and Scandinavia, the use of coking coal, limestone, oxygen and electricity for refining, also energy for the hot and cold rolling processes and finally fabrication. However, this is significantly ameliorated by the recycling of scrap metal.

Steel can be repeatedly recycled without any degradation of properties or performance, and globally some 40% of all steel is manufactured directly from recycled material. In the UK approximately 94% of all steel within deconstructed buildings is recycled, and up to 99% of steel in structural sections is reportedly recycled. A recent enhancement towards the recycling of structural steel is the stamping of sections with their size and steel grade to allow easy identification and reuse on deconstruction, particularly when bolted rather than welded on site.

The data from the International Iron and Steel Institute, listed in **Table 4**, take into account the 'life cycle' considerations for steel and assumes that 85% will be recycled when the construction is demolished. For comparison with other construction materials and systems, both embodied energy and CO_2 emissions per tonne of steel are recorded.

Tonnes CO_2 per tonne of steel	1.01 T
Embodied energy per tonne of steel	16837 MJ

Note: To make an appropriate comparison between materials the mass of alternative materials and the resultant effects on the whole building structure including foundations must be analysed.

Table 4 Embodied energy and CO_2 emissions per tonne of steel

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Chapter 41

Aluminium

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The production, fabrication processes, surface finishing and properties of aluminium alloys used in construction are described, together with notable examples of construction using aluminium products. Also, sustainability issues covering environmental, economic and social aspects are outlined.

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Introduction

The properties of aluminium alloys – high strength, low weight, good formability, good corrosion resistance, ease of finishing, joining and fabricating and excellent recyclability – have made aluminium a popular worldwide choice in building and construction applications, as well as in transport, packaging and general engineering.

In terms of weight percentages, aluminium is the third most common element in the Earth's crust, at 8%; only oxygen and silicon exist in greater percentages. However, aluminium is not found in the metallic state but in combination with oxygen and silicon as stable, complex alumino-silicates. The primary production process for aluminium metal is an electrolytic one and the aluminium industry had to wait for Michael Faraday, in the 1830s, to develop the rules for the generation of electricity before the commercial production of aluminium could begin. The industry effectively began in 1886 with the development of the electrolytic process patented simultaneously by Charles Hall in the USA and Paul Heroult in France. Although their process has been engineered out of all recognition, the principles used remain the same today. From humble beginnings, the aluminium industry has developed to the point where some 30 million tonnes of primary aluminium are produced worldwide each year, a tonnage predicted to grow to 50 million tonnes over the next 20 years. This has made aluminium the most widely used non-ferrous metal, the tonnage exceeding that of all of the other non-ferrous metals added together.

The ore most commonly used for the production of metallic aluminium is bauxite, an alumino-silicate found in conjunction with the oxides of iron, silicon and titanium. About 40% of the world's primary aluminium begins life as bauxite, mined in Australia by open-cast mining. Aluminium oxide, alumina, is extracted from bauxite by a caustic soda ingestion process, known as the Bayer Process, fortunately developed very shortly after the 1886 work of Hall and Heroult. The finely divided aluminium oxide is dissolved in a bath of molten cryolite (Na_3AlF_6) at a temperature of 950°C. The cryolite forms the electrolyte of the cell; the consumable anode and permanent cathode are both made of carbon. The cells run at a low voltage but very high amperage, typically 200 000 amps. Each cell in a primary smelter produces about 1 t of pure metal per day. The production of 1 t of metal requires 2 t of aluminium oxide, which in turn requires 4 t of bauxite. Primary smelters across the world are now operating at more than 500 000 tonnes per year and 1 million tonnes per year smelters are now being built.

The primary production process requires large amounts of electricity. Modern smelters now operate at about 13 kWh for each kilogram of metal produced. This investment in energy is more than repaid by energy savings of the aluminium in use and energy saved by recycling after use. The use of hydroelectricity is widespread in the primary aluminium industry, reducing greenhouse gas emissions. Environmental protection systems are installed at primary smelters to ensure that emissions, gaseous, liquid or solid, are minimised.

The product of the primary process is pure aluminium, of the order of 99.7% aluminium, with small amounts of silicon and iron, the purity depending, among other things, on the age of each electrolytic cell. Metal of this composition has limited use and the pure metal is alloyed with other metals to produce the multitude of alloys that are used industrially.

Aluminium alloys

A distinction must be made between the wrought alloys, those that will be rolled, extruded and forged, and the cast alloys, for which a wide variety of casting processes can be used.

Dealing first with the wrought alloys, these account for the greater majority of aluminium alloy production.

Alloy designation	Major alloying element	Work hardening	Heat treatable	Typical uses
1XXX	None	✓		Foil, electrical, signage
2XXX	Copper		✓	Aerospace and high-strength applications
3XXX	Manganese	✓		Beverage cans, architectural sheet
4XXX	Silicon	✓		Welding filler rod
5XXX	Magnesium	✓		Marine and construction sheet and plate
6XXX	Magnesium and silicon		✓	Architectural and transport extrusions
7XXX	Zinc		✓	High-strength plate and extrusions (Al–Zn–Mg)
8XXX	Others, e.g. tin, lithium			Bearings (Al–Sn) High-strength plate (Al–Li)

Table 1 International four-digit system for wrought aluminium alloys

There is a universal four-digit description of the wrought aluminium alloys that classifies the alloys according to the major alloying element used. This is illustrated in **Table 1**. The pure aluminium alloys, containing at least 99.0% aluminium, form the 1000 series alloys. This is effectively aluminium of various degrees of purity, with the last two digits giving an indication of the purity. Thus, alloy 1080 contains 99.80% of aluminium, 1050 contains 99.50% and 1200 contains 99.00%. These alloys are the widely used packaging materials such as household foil and are also used for electrical applications: aluminium on a weight-for-weight basis being a better electrical conductor than copper. The 2000 series alloys are alloys where the principal alloying element is copper, up to about 5%. These alloys can develop high strengths by heat treatment and are widely used in aerospace applications. The 3000 series alloys have manganese as the major alloying element, at levels of up to 2%, and are used as sheet alloys in packaging, beverage cans, etc., and in building and construction. The 4000 series alloys contain silicon and these alloys are largely used as welding filler rod material. The 5000 series alloys contain magnesium and they can be rolled, forged and extruded, with good strength and corrosion resistance, useful in many marine applications for example. The 6000 series alloys, containing both silicon and magnesium, are probably the most versatile of the whole series, being the most common extrusion alloys but also widely used as sheet and plate, as well as forgings. The 7000 series alloys contain zinc and, in conjunction with magnesium in the alloy, very high tensile strengths can be developed by heat treatment, leading to their use in aerospace for example. The 8000 series alloys are those alloys that do not fit easily into any of the 1000 to 7000 series, with a principal alloying element not being one of those listed above.

While the wrought aluminium alloy description is used and recognised internationally, the same is not true for the cast alloys where national standards have predominated, although in the EU the CEN standards are now becoming more widely accepted. The cast alloys are largely based on the aluminium–silicon system, these alloys having very good casting characteristics. Silicon contents up to 12% are common but these alloys are not produced with bright finishes and the Al–5% Mg alloy is available with good finishing characteristics but can only be cast in simple shapes, used in door furniture for example. The cast alloys commonly used in construction are shown in **Table 2**.

The mechanical properties of the wrought alloys can be changed by cold deformation, by cold rolling, drawing, etc. As the amount of deformation increases, the tensile strength increases but the ductility is reduced. For some of the wrought alloys, the 1000, 3000 and 5000 series alloys, the only control that the producer has over mechanical properties is by cold deformation and partial or full

Standard BS EN 1706	Standard BS 1490 (obsolete)	Production method	Characteristics
AC-44100 AlSi12(b)	LM6	Sand and permanent mould castings	Used in as-cast condition, excellent corrosion resistance and castability
AC-42000 $AlSi_7Mg$	LM25	Sand and permanent mould castings	Can be heat-treated for better tensile properties
AC-51300 $AlMg_5$	LM5	Mainly sand cast but permanent mould and simple die castings can be produced	Excellent corrosion resistance and can be bright anodized

Table 2 Cast aluminium alloys commonly used in construction

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Alloy	Standard	Temper	0.2%PS*: N/mm^2	UTS*: N/mm^2	Elongation*: % min
1080A (sheet)	BS EN 485 Part 2	O	15 min	60 min	26–35
		H18	105 min	125 min	2
3103 (sheet)	BS EN 485 Part 2	O	35 min	130 max	17–28
		H18	165 min	185 min	1–2
5083 (sheet and plate)	BS EN 485 Part 2	O	105–125 min	255–275 min 350 max	11–16
		H24	250 min	340 min	4–8
6063 (extrusion)	BS EN 755 Part 2	T5	130 min	175 min	8
		T6	170 min	215 min	8
6082 (extrusion)	BS EN 755 Part 2	T5	230 min	270 min	8
		T6	250–260 min	290–310 min	8–10
AC-44100 (casting)	BS EN 1706	As-cast, sand	70 min	150 min	4
		As cast, permanent mould	80 min	170 min	5
AC-42000 (casting)	BS EN 1706	As-cast, sand	80 min	140 min	2
		As-cast, permanent mould	90 min	170 min	2.5
		T6 sand cast	180 min	220 min	1
		T6 permanent mould	220 min	260 min	1
AC-51300 (casting)	BS EN 1706	As-cast, sand	90 min	160 min	3
		As-cast, permanent mould	100 min	180 min	4

Note: * Where a range is given this is because the minimum specified value is dependent on the gauge of the material specified.

Table 3 Mechanical properties of commonly used cast and wrought alloys

annealing. Cold rolling is the predominant process and these alloys are available as sheet and plate products, although they can also be extruded if required. In sheet form, the tempers range from the annealed condition, referred to in Standards as the O condition, and thereafter as H12 up to H18 as the material is work hardened. **Table 3** shows some of the alloys commonly used in building and construction from the 1000, 3000 and 5000 series, together with the tempers and mechanical properties. For architects and designers, the 0.2% proof stress and the ductility are more important than the ultimate tensile strength (UTS) since these properties will determine the fabrication characteristics and the behaviour under load.

The 2000, 6000 and 7000 series alloys have alloy compositions that allow mechanical properties to be enhanced by heat treatment. Since the 6000 series alloys, Al–Mg–Si, are much more widely used in building and construction than the others, particularly as extrusions, then this series of alloys and this process will be used to describe the fabrication process and the subsequent heat treatment.

The versatility of aluminium as a metal is complemented by the versatility of the extrusion process. Few other materials, metallic or non-metallic, can be extruded with the ease of aluminium alloys. The extrusions are produced by heating aluminium alloy billets to about 500°C and forcing hot metal through shaped holes cut in a steel die, the profile of the hole being that required of the extrusion. As the extruded section emerges it is cooled and cut to the desired length. Heat treatment is then used to optimise the material's inherent mechanical properties. Computer-aided design and manufacture are used to reduce design time, improve tolerances and control the production process closely. Intricate shapes can be extruded, with great dimensional accuracy and good surface finish, reducing fabrication and machining costs. Structural efficiency is possible, placing metal where it adds strength and omitting it where it is not needed.

In the 6000 series alloys, Al–Mg–Si, containing up to 1.3% each of magnesium and silicon, some of the magnesium and silicon is in solid solution in the aluminium crystal structure but most of it exists at room temperature as an intermetallic compound, Mg_2Si. The particle size and distribution of the Mg_2Si determine the mechanical properties of the alloy. If, immediately after extrusion, the section is cooled very quickly, by air blast for example, the magnesium and silicon can be held in solid solution and subsequently precipitated out by an ageing treatment at about 160°C. This allows Mg_2Si to form as finely divided particles that increase the strength with no detriment to the ductility. This temper is referred to in the Standards as T5. A full solution heat treatment followed by quenching and precipitation is also possible, namely T6 temper, but the T5 temper is more common for architectural sections, see **Table 3**.

Of all of the aluminium castings produced, some 75% are used in transport applications. Never the less, there are good examples from building and construction of cast aluminium being used as panels as a cladding for an office block, staircases and street furniture such as bollards and handrails. Usually the castings are in the uncoated condition and they weather well, Eros being a good

example, an aluminium casting in place in Piccadilly Circus, London, since 1893. The castings are usually produced as sand or permanent mould castings, depending on the number required. The mechanical properties of some of the cast alloys are shown in **Table 3**.

Some mention should be made of the behaviour of aluminium products in a fire, since a large amount of misinformation arose from the Falklands War over the loss of nine ships and whether aluminium had played some part in the loss by 'burning'. Aluminium is widely used in the superstructure in shipping because of its good mechanical properties and low density, keeping the centre of gravity as low as possible. Of the nine ships lost, three had an aluminium superstructure. The loss of HMS *Sheffield* was particularly quoted in the concerns in the media about aluminium although it should be noted that the HMS *Sheffield*, as its name suggests, was an all-steel vessel. A subsequent Ministry of Defence investigation concluded that no vessel was lost as a result of having an aluminium superstructure nor was there any evidence that aluminium had burned (Secretary of State for Defence, 1982).

The thermal conductivity of aluminium is about four times that of steel and the specific heat twice that of steel. Therefore heat is conducted away faster and thus a greater heat input is required to bring the same weight of aluminium to a given temperature, compared with steel. When an aluminium structure is exposed to the heat of a fire, the high thermal conductivity means that the heat will be conducted rapidly away from the exposed area. This reduces hot spots, extending the serviceability period but also causing the temperature to rise elsewhere. The extent of the dissipation will depend on the degree of thermal insulation used with the aluminium in the structure. Weathered aluminium has a much higher reflectivity than steel or stainless steel, again assisting in prolonging the endurance in a fire. Aluminium melts at 660°C and the aluminium alloys at similar temperatures. When the ambient temperature rises, aluminium in the massive form will show an increase in temperature and will rapidly lose strength at temperatures in excess of 200°C. At its melting point the aluminium will start to melt. It does not burn. The relatively low melting point of an aluminium roof, for example, significantly lower than that of a steel one, is an advantage since if the roof material melts at relatively low temperatures in a fire then the fire is vented and the temperature rise inside the building is restricted. There are examples of the damage in a fire being minimised in this way, whereas with a steel or ceramic type of roof the temperature rise in the building will magnify the damage.

Aluminium finishing processes

With good design, aluminium can be used in building and construction with confidence about the service life and corrosion behaviour. The San Gioacchino Church in Rome offers a good example of the longevity of aluminium, the dome was clad in aluminium in 1897 and that cladding is still there today in pristine condition, with very little maintenance. Nearer to home, the statue of Eros, cast in aluminium and erected in 1893, has withstood the London atmosphere without any problems. At the end of World War II aluminium aircraft were recycled into prefabricated housing, known as prefabs, which were supposed to be temporary but some of them are still in good use today and have been given 'listed building' status.

The cast and wrought aluminium alloys used in building and construction applications have good corrosion resistance, particularly in the pH range 6.0–8.5 but the designer must be aware of the mechanisms of corrosion so that the design minimises the extent of corrosion. The designer must also be aware of the finishing processes, such as painting and anodising, which can be applied, not only to reduce corrosion but also to enhance the appearance of the aluminium structure. The uncoated aluminium article always has a thin adhering layer of aluminium oxide on the surface that helps to improve the corrosion behaviour. In normal atmospheres, therefore, corrosion is usually pitting corrosion, the rate decreasing with time. The surface appearance after some time is then a dull grey colour but the rate of attack slows as the pits fill with corrosion debris. The design, however, must allow for any rainfall to run off the sheet or extruded section so that contact time is minimised.

Accelerated corrosion can occur when dissimilar metals are joined so that, in the presence of an electrolyte – rainwater or seawater for example – an electrolytic cell is established. Since aluminium will often be the anode of such a cell, with other metals such as copper and steel being the cathode, the aluminium component will suffer accelerated corrosion. When using aluminium in building and construction, therefore, the designer must avoid such galvanic couples, for example by separating aluminium roofing sheet from steel fixings, screws, etc. by using non-conducting neoprene washers or using aluminium fixings. Electrical contact between aluminium and stainless steel will only slightly increase the corrosion of aluminium in dry atmospheres but it will be markedly increased in moist atmospheres, particularly in marine conditions.

Where the preservation of the surface appearance is desirable and where a colour finish is required, aluminium can be treated with a variety of finishes, including mechanical and chemical finishes, plating, anodising and organic coating. The most widely used processes are anodising and painting, these processes being summarised in Seddon (2004).

Anodising is an electrolytic process that produces a dense, chemically stable, protective aluminium oxide film that is an integral part of the underlying aluminium; it cannot peel or flake off, see **Figure 1**. Most films produced

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Figure 1 The M4 Severn Bridge with anodised aluminium baffles to deflect the wind, 1996 (courtesy of the Aluminium Federation)

by anodising are translucent and so show the silvery look of the aluminium but other colours are achievable by a variety of anodising process variations. All of the aluminium alloys can be anodised but not all of the alloys are recommended for the process. The Al–Si alloys take a grey colour on anodising that is protective to corrosion but not visually attractive. On the other hand, some bright trim alloys can be anodised with a mirror finish. Prior to anodising, the surface must be of good quality and free of any grease or other detritus. This is achieved by a caustic dip and rinse. The aluminium piece is then made the anode of an electrolytic cell, using sulphuric acid as an electrolyte and aluminium as the cathode. The oxide thickness will vary from 5 to 25 microns; 25 micron thickness is typically chosen for outdoor applications and 10–15 microns is usual for indoor applications. For decorative trim the thickness is 5 microns. The anodic film can be coloured by dyeing using any colour that the designer requests. After anodising, the anodic film is sealed using boiling water. Integral colour anodising processes, as an alternative to dyeing, can be used; colours ranging from gold to bronze to black can be incorporated into the oxide film using nickel and cobalt salts. Alternative anodising-type processes such as Keronite are now being developed. The cost of anodising is very dependent on the surface area and therefore the process lends itself more to extrusions than to sheet, which can be anodised but are more often painted.

Of the paint films available, polyester coating is the most widely used. Extrusions are pre-treated to remove any grease and debris and then passed to a spray booth. The pigment and resin are carried by compressed air, with an electrostatic spray gun to ensure good adhesion. The coated sections then pass to a curing oven where the resin sets.

Powder coating is the less costly of the two processes and the colour range available is extensive and a variety of gloss levels can be chosen. Designers should be aware, however, that colour changes are not straightforward and white is a preferred option in most cases, unless there is a substantial quantity to coat. Powder coatings properly applied can be expected to last for up to 30 years and guarantees can be provided with lives of 15 to 30 years. The guarantee will probably have conditions that specify regular maintenance and cleaning by washing, **Figure 2** showing a good example of coated aluminium extrusions used to fabricate a conservatory. The more expensive anodised films should give longer lives, 40 to 50 years, and they have better wear resistance, the surface aluminium oxide being very hard and durable. They are also less likely to suffer from fading of the colours. Again, regular maintenance and washing pay dividends in enhancing the service life.

Sheet aluminium can be processed as whole coils to produce coil-coated painted material that will go to be used in roofing, siding, flashing and guttering.

Figure 2 Powder coated aluminium extrusions used to fabricate a conservatory (courtesy of Sapa Building Systems Ltd)

Examples of the use of aluminium products in construction

Construction is a major end-use for all aluminium products, where, in the UK, a total of over 150 000 t are used each year, almost all of it extruded and rolled with some castings. For extrusions, the window market, both new and replacement, is the most important sector, where the good mechanical properties of the aluminium alloy combine with the extrudability that allows the designer to incorporate complex shapes with grooves for joining, draught excluders, glazing beads and thermal break to improve the thermal characteristics, **Figure 3** showing a good example. Other applications for extrusions are:

- curtain walling and structural glazing
- prefabricated buildings such as conservatories and greenhouses
- heating and ventilation
- shopfronts
- signage
- architectural hardware such as balustrading
- scaffolding, steps and ladders
- partitions
- components for structures such as bridges.

The alloys used are usually from the 6000 series and the extrusions are in service, both indoors and out, in the mill finish or painted or anodised conditions.

The aluminium rolled products are equally versatile. The sheet can be roll formed so that it is ideal for roofing and siding applications. Aluminium coils can be roll formed on site to any desired length, which means that flashings and guttering can be produced to be seamless, without the need to transport very long lengths to the site. In heating, ducting and ventilation, advantage can be taken of aluminium's good physical as well as mechanical properties.

Figure 3 Cut-away section of a complex aluminium window extrusion section (courtesy of Sapa Building Systems Ltd)

Figure 4 Aluminium roofing on the Wimbledon No. 1 Court, 1997 (courtesy of the Aluminium Federation)

The aluminium extruder or roller, the anodiser or coater, the stockist involved in the supply chain and the installer will all have taken great care to protect the surface of the aluminium during storage, transport, delivery and installation, particularly where the end-product will be used in critical applications such as shop fronts and signage. Shrink wrapping and paper interleaving are available and the surface can also be protected by plastic coating, which for rolled products can act not only to protect the surface but also as a lubricant during any fabrication process. Wet cement and concrete have a high pH and splashing during installation of aluminium products should be avoided.

Very complex roofing installations can be computer designed and installed using aluminium sheet (*Roofing*, 2006). Examples include the new Number 1 Court at Wimbledon by Corus Building Systems, **Figure 4**, and the Manchester Stadium built for the 2002 Commonwealth Games, **Figure 5**. The light weight of the roofing, per unit of area, means that a retractable roof can be incorporated into the design so that spectators and participants are not subject to the vagaries of the weather, the Millennium Stadium in Cardiff being a good example, **Figure 6**.

In some applications, extrusions and rolled products can be used together. A notable example is the NatWest Media Centre at Lord's Cricket Ground, **Figure 7**. This building, the world's first aluminium semi-monocoque building, was designed by the architectural practice Future Systems. In this application, aluminium is being used structurally as well as for fitting out the building. Alloy 5083 plate was

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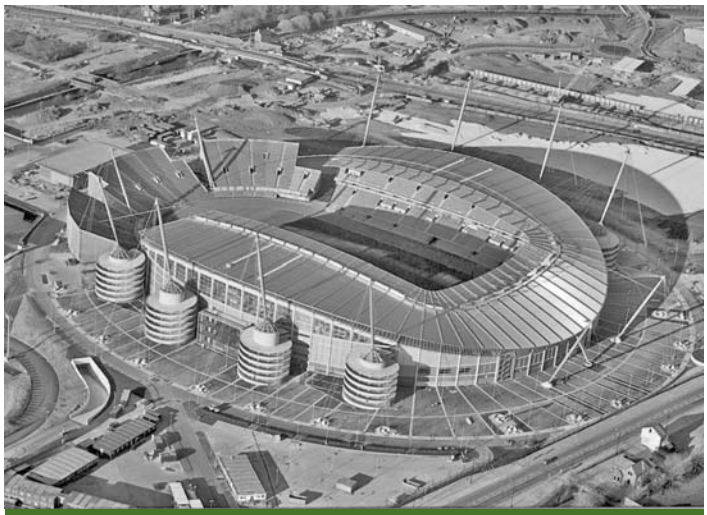

Figure 5 Aluminium roofing on the Manchester Commonwealth Games Stadium, 2002 (courtesy of the Aluminium Federation)

fabricated in a West Country shipyard and taken by road in five large fabrications to Lord's Cricket Ground where the building was erected on concrete foundations in 1999. The result is a magnificent example of how versatile the aluminium products are and how well aluminium and glass can work together. A further example of aluminium working well with other materials, in this case oak and glass, is seen in **Figure 8**. This small pedestrian bridge, the Bridge of Aspiration, joins the Royal Ballet School to the Royal Opera House, in Floral Street, Covent Garden, at the fourth-floor level. It was designed by the Wilkinson Eyre architectural practice using aluminium box beams. The twisted geometry was necessary because the buildings had to be joined at slightly different levels and the openings were offset by a small distance (Wilkinson Eyre Architects website). A further example of the versatility of aluminium is seen in **Figure 9**, the Salvation Army HQ Building in Millennium Walk, London, with a structurally glazed aluminium curtain wall with vertical glass louvers to the main elevation.

Figure 7 Aluminium plate, sheet and extrusions were used to fabricate the NatWest Media Centre at Lord's Cricket Ground, 1999 (courtesy of the Aluminium Federation)

As well as being used for 'new build', aluminium products are widely used in the renovation of old buildings in preference to their demolition. This practice is both economic and also a contributor to reducing the 'carbon footprint' of the building. Many tower blocks of the 1950s and 1960s have recently been given a new lease of life by the use of aluminium cladding, preventing spalling of the concrete, and by the installation of replacement aluminium windows. An example is Newcastle House, Nottingham, a listed building, shown in **Figure 10**, with replacement aluminium windows installed by Glostal in

Figure 6 The Millennium Stadium, Cardiff, with retractable aluminium roof, 1999 (courtesy of the Aluminium Federation)

Figure 8 Aluminium extrusions were used in the Bridge of Aspiration, Covent Garden, London, 2003 (courtesy of the Aluminium Federation)

Figure 9 Salvation Army HQ, Millennium Walk, London. Aluminium curtain wall (courtesy of http://www.a2n.co.uk)

Figure 10 Newcastle House, Nottingham. Replacement aluminium windows (courtesy of www.a2n.co.uk)

the 1980s, replacing the original steel frame windows. The Council for Aluminium in Building (CAB) recently held the Aluminium in Renovation Awards 2007. Some 40 restored buildings were entered for the competition, all of which became an asset to their surroundings (Council for Aluminium in Building, website).

Recycling of aluminium used in construction

Quite rightly there is a great deal of interest today in the recycling of all materials. It is worthy of note that the aluminium industry has had a recycling industry since the 1920s, the driver in the early days being the economic advantages. Aluminium has a recycling characteristic found in very few other materials used in building and construction in that closed-loop recycling operates. If 1 t of recycled aluminium windows are returned after use to the aluminium industry then, allowing for small melt losses and using best technology in the secondary refiner, 1 t of extrusion billet can be cast and extruded into the same alloy and section as the original material. Similarly, with recycled aluminium roofing and siding sheet or aluminium cast street furniture, they can be made back into the same alloy and the same product.

The drivers for recycling are now, of course, more extensive than just the economic one, although it remains important that the aluminium recycling industry is still economically viable and needs no subsidy. Recycling aluminium saves raw materials, saves energy, reduces emissions compared with primary production and saves on landfill space. The energy equation here is of great importance. It was noted in the 'Introduction' that the primary production of aluminium is energy intensive but compared with primary production, on a weight-for-weight basis, aluminium can be produced by the recycling route for an energy consumption of only 5% that of primary production. Not only that but the recycling process can be repeated time and again with no degradation of the material, so that closed-loop recycling is possible.

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The other important consideration that designers and architects must bear in mind is environmental legislation on recycling from the European Parliament. As far as aluminium is concerned, there is legislation covering aluminium recycling targets for transport (End of Life Vehicles Directive), packaging (Packaging and Packaging Waste Directive) and electronic goods (Waste Electrical and Electronic Equipment Directive). One can anticipate that recycling targets for materials used in building and construction are already being planned and, since the life of a building is considerably longer than that of a car, a beverage can or a television set, then designers should now examine the implications for their choices of materials even if recycling targets are not yet on the statute book.

Architects and designers often ask for details of the recycling rate for aluminium used in construction. It is necessary to define the recycling rate so that valid comparisons can be made with competitive materials. During the production and fabrication of aluminium products, aluminium scrap is produced that the aluminium industry recycles in-house. This material is known as 'new scrap' and the recycling rate is 100%. There is no problem with collection; it is in the hands of the industry and it is of a known alloy and has a known high value. It is all recycled. The legislators and the end-users are more interested in the recycling rate of the aluminium that is used in construction and then becomes available for recycling, perhaps 40 years later when the building is redundant. This material is known as 'old scrap'. The act of recycling old scrap now has to take account of two processes: recovery and refining–recovery by the demolition company and refining by the aluminium industry.

In 2003 the European Aluminium Association commissioned an independent study by the University of Delft to determine the recycling rate of old scrap from building and construction (van Houwelingen, 2004). Across Europe, buildings – commercial, residential, industrial and sporting stadia – about to be demolished, have been studied to determine the total aluminium content in the building. After demolition the company responsible for the demolition was asked to provide weighbridge tickets for any aluminium collected and sold into the scrap metal industry for recycling. In the UK the old Wembley Football Stadium was one of the buildings studied. The results showed that recycling rates of 92–98% were achieved for aluminium used in building and construction. Having regard to the current price of baled old-rolled aluminium scrap, of £700 per tonne, these results should come as no surprise. Demolition companies do not send such valuable material to landfill when the aluminium industry is ready, willing and able to recycle it. For the architect who originally selected aluminium for the building, that material during the life of the building has been maintaining its intrinsic value and, on demolition, the scrap price per tonne is probably exceeding the original purchase price of the aluminium product.

This successful recycling rate has led to a conundrum because designers will sometimes request that their aluminium product, to be used in a construction contract, is made only from recycled material. It must be remembered that the aluminium product can only be recycled at the end of the useful life of the article being used, a roof, a window and so on. Aluminium in construction may have been put into service some 40 years or more ago. This is the material now becoming available for recycling 40 years later but during that 40-year period the amount of aluminium used in construction has grown twenty-fold. There is therefore a limited supply of aluminium from construction applications available for recycling and the demand for recycled aluminium may well exceed the supply.

In summary, the aluminium industry has an enviable record on recycling from all end-uses, building and construction, transport, packaging and general engineering. There are huge energy advantages for the industry, many environmental advantages for the world at large and economic advantages at the end of the life of the building for the owner.

Sustainability of aluminium used in construction

The measurement of sustainability is not always understood. Accepting the commonly used definition 'meeting the needs of the present without compromising the ability of future generations to meet their own needs', how does aluminium as a construction material measure up to this yardstick? Since sustainability has economic, environmental and social aspects it is necessary to develop indicators that the industry can use regularly to measure the progress towards sustainability. The indicators must be fairly chosen and not merely be measures that put the industry in the best light. The aluminium industry has worked with all of its stakeholders, employers, employees, investors, end-users, suppliers, legislators, NGOs and universities, to decide on the indicators that should be used. The indicators must be quantifiable and reproducible so that comparable results can be obtained. Since society continually moves the sustainability 'goalposts' towards higher targets, no industry can claim more than to be moving towards a sustainable position or using a sustainable material.

In the environmental area, the aluminium industry can point to the almost unlimited supply of ore, bauxite, known reserves covering current production levels for 300 years. At the end of the life of a bauxite mine there is a mine restoration programme (International Aluminium Institute, 2004). The constraint is in the energy required

in primary production, which is more than offset by energy savings in use and by recycling. In complying with all environmental legislation in the EU on production and fabrication, the industry has made major savings in energy consumption and emission levels, solid, gaseous and liquid. The high recycling rates, the fact that there is no material degradation on recycling and the ability to operate closed-loop recycling, are major environmental advantages over competing materials.

From the economic viewpoint, the aluminium industry started at zero production in 1886 and now has a worldwide primary capacity of about 30 million tonnes per year. Over the next 20 years this is expected to be at 50 million tonnes as new smelter capacity comes on stream in China, India, the Middle East, South America, etc.

Investment in new capacity and in research and development in the industry is at a high level. The primary metal price worldwide, determined largely by the London Metal Exchange daily price, has fluctuated in a way that has not always been to the industry's advantage, fuelled often by the large financial institutions which, although not being end-users, see aluminium as an investment opportunity. A good linear relationship has been shown between GDP per head of population and the usage of aluminium, expressed as kg per head in any country (European Aluminium Association, 2006). In the wealthy countries of the Western world, aluminium consumption reaches 30 kg per head per annum but with much lower consumption in the developing countries with low GDP. As these countries increase their GDP per head, the aluminium industry confidently predicts that their consumption of aluminium will grow accordingly. Hence the industry and its investors feel sure that the growth in primary capacity will be matched by the growth in consumption in all markets.

In a social context, the aluminium industry invests heavily in industry training and the wage levels in any geographic area compare favourably with other industries. Health and safety issues are taken seriously and are rigorously monitored. The aluminium industry acts as a 'good neighbour' and industry surveys show increasing local involvement in both financial support and man-hours given. Society in general has a ready acceptance of aluminium products.

Summary and future trends

Aluminium is found in nature as a complex alumino-silicate, known as bauxite. From this, ore aluminium oxide is extracted which is then used in an electrolytic cell to produce aluminium metal. The metal is alloyed with various other metals to produce a series of alloys which have attractive mechanical and physical properties. These properties can be varied by mechanical processing, such as cold rolling, or by heat treatment. The aluminium alloys are easily fabricated into extruded and rolled products, forgings and castings which are widely used in building and construction, transport, packaging and general engineering.

Although the corrosion resistance of the aluminium alloys is generally good, in many construction applications they are given surface protection and finishes that enhance the appearance and improve the lifetimes. The thermal properties mean that damage by fire can be minimised.

After use, at the end of the life of the building or construction, aluminium has very high recycling rates, up to 98% and the material can be subjected to closed-loop recycling. Aluminium is a sustainable engineering material, confirmed by applying sustainability indicators that cover economic, environmental and social aspects. The industry is confident that the great expansion now taking place in primary aluminium capacity will be matched by the growth in use of aluminium across the world and across all of its markets. It is widely used in stadia and the 2012 Olympic Games in London presents a great opportunity for the use of aluminium products.

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Further reading

The aluminium industry in the UK, Europe, the USA and internationally supports many trade associations whose websites carry a great deal of technical information on the use

of aluminium in construction, as well as lists of members of each association who can supply products and services. The major associations are listed below.

Aluminum Association (USA) http://www.aluminum.org.

Aluminium Federation (UK) http://www.alfed.org.uk.

Aluminium Finishing Association (UK) http://www.afauk.org.uk.

Aluminium Stockholders Association (UK) http://www.asauk.co.uk.

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Council for Aluminium in Building (UK) http://www.c-a-b.org.uk.

Diecasting Society (UK) http://www.dcsoc.org.uk.

European Aluminium Association (Europe) http://www.eaa.net

GDA (Germany) http://www.aluinfo.de.

International Aluminium Institute (Worldwide) http://www.world-aluminium.org.

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Chapter 42

Copper

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The properties and applications of copper and copper alloys, such as brass, tin bronze and aluminium bronze, used in construction, pipework services, architectural metalwork and electrical applications are discussed. European Standards for materials and products are covered, together with an appreciation of the sustainable nature (including recyclability) of copper and copper alloys.

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Introduction

Copper is a metal that is naturally present in the Earth's crust and it is essential to the development of all forms of life. It is the oldest metal used by man: the first copper coins date from 8700 BC. Together with tin it forms bronze, the first metallic alloy produced by man from around 2300 BC, a period that became known as the Bronze Age. At the turn of the tenth century BC, the legendary wealth of King Solomon was said to have been generated from the operation of mines, rich in copper, in the Sinai desert.

The history of copper is closely linked to that of Cyprus, because it was on the island of Cyprus that the first open-pit copper mines were operated, enabling the Hellenic, Mycenaean and Phoenician civilisations to prosper. They organised the red metal's trade in the Mediterranean, although the Romans called it *aes cyprium* (literally: 'metal from Cyprus'). Over time, the term changed and became 'copper'.

Sources of copper supply

Copper is produced from three sources:

- The extraction and processing (refining) of the copper ore, called 'primary production'.
- The recycling of end-of-life products, called 'secondary production'.
- The direct recycling of 'new scrap' (waste resulting from the manufacturing process).

Copper mining and production

Copper minerals are widely distributed throughout the Earth's crust, typically being copper sulphide, copper iron sulphide and copper oxide. To be economically viable, the copper content of an ore must contain at least 0.2% copper. The largest copper mining country by far is Chile, which produces about 35% of the world's copper, followed by the USA with 8%. Other leading mining countries are Indonesia, Peru, Australia, China and Russia. The global mining production for 2006 was about 16 million tonnes (**Figure 1**).

Most ores are mined by the open pit method, where ores near the surface can be quarried after removal of the surface layers. An open pit mine is shown in **Figure 2**. The size of the huge trucks gives an idea of the scale.

When an ore containing about 0.2% copper is processed to produce copper of 99.9% purity, clearly a lot of waste rock and impurities, such as sulphur and iron, have to be removed. This is achieved by physical (grinding to powder and concentrating by froth floatation), and then chemical methods (roasting in air, smelting and converting in air).

Air (oxygen) plays an important role in oxidising the impurities, such as iron and sulphur, to acceptable levels. However, oxygen has little effect on conductivity and the highest electrical conductivity coppers (e.g. CW004A) contain a small percentage of oxygen (typically 0.04%).

For copper which has to be fabricated by brazing and welding, this small percentage of oxygen may cause problems, so it is removed during the production process by the addition of phosphorus, a deoxidiser (phosphorus content in the range 0.001–0.06%). The phosphorus deoxidised copper has 98% of the conductivity of pure copper and is used in plumbing, building services, roofing and cladding. The material designation is CW024A.

Recycling

Recycling of copper limits the use of natural resources, reduces energy consumption and decreases waste disposal problems. According to the most recent global figures (ICSCT, 2006), 37% of the world copper usage comes from recycling (in Europe, this figure is 42%), including

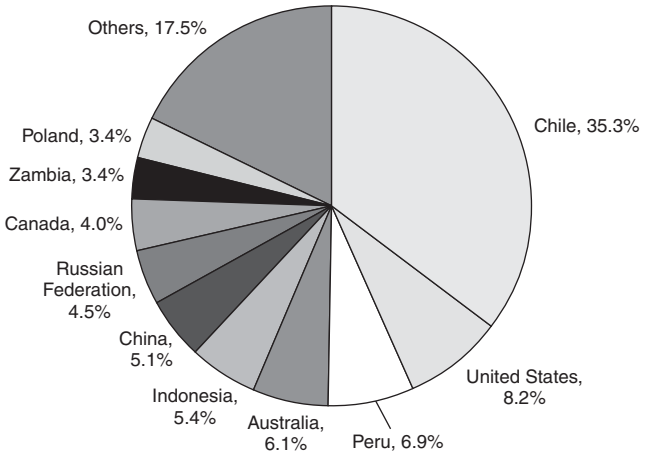

Figure 1 World copper production by country

Figure 3 Recycling copper scrap (courtesy of Ampthill Metal Company)

both secondary copper production and the direct recycling of 'new scrap' (**Figure 3**). Copper is easily recycled without loss of properties, ad infinitum.

Copper alloys

Copper alloys are described generically by such terms as brass, bronze, copper-nickel and copper-nickel-zinc (which are called nickel silvers because of their shiny white colour). The properties of copper alloys, which occur in unique combinations found in no other alloy system, include high thermal and electrical conductivity, a wide range of strengths, excellent ductility and toughness, as well as good corrosion resistance in many different environments.

Designations

Coppers and copper alloys are produced to conform to a wide variety of national and international specifications, prepared to suit differing conditions and requirements. All manufacturers and good stockists are approved to supply material, if required, under the quality assurance requirements of the BS EN ISO 9000 series.

The material designation for copper and copper alloys is a six-character, alpha-numeric series, which begins with C. The second character indicates the product form. The third, fourth and fifth characters are digits that allow precise identification of the alloy. The sixth character indicates the alloy group.

Figure 2 Bingham Canyon open pit mine

Grades

The grades of copper used in building wire require the highest electrical conductivity. This is met by either Cu-ETP (CW004A) which has 0.02 to 0.04% oxygen or Cu-OF (CW008A) which is oxygen free. The grade of copper used for tube and sheet is phosphorus deoxidised copper (CW024A). See **Table 1** for properties.

Forms

Copper is refined from ore and shipped to fabricators mainly as cathode, wire rod, billet, cake (slab) or ingot. Through extrusion, drawing, rolling, forging, melting or atomisation, fabricators form wire, rod, tube, sheet, plate, strip, castings, powder and other shapes. These copper and copper alloys are then shipped to manufacturing plants where they are used to make products to meet society's needs.

Properties

Typical mechanical and physical properties of phosphorus deoxidised copper CW024A (C106) used in building are shown in **Table 1**.

The following points show why copper is an ideal material for numerous building and construction applications:

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Melting point: °C	Density: g/cm^3	Coefficient of expansion: °C	Thermal conductivity: W/mK	Tensile strength: N/mm^2	Elongation: %	Hardness: HV
1083	8.94	18×10^{-6}	390	220–385	40–55	40–110

Table 1 Typical properties of CW024A

1 The high melting point of copper means that:
   - it will maintain its integrity in a fire, although the strength will be at the lowest level shown in the table;
   - in-service copper will not creep (flow) like some metals and plastics.
2 The high thermal conductivity value means that any heat built up in a fire is quickly dissipated.
3 The coefficient of expansion is approximately 40% less than zinc and lead, which results in less thermal movement.
4 Copper sheet can be specified by its tensile strength and will be supplied in either the soft (R220) or 1/2 hard (R240) condition. If further fabrication is carried out, such as pressing, bending or hammering, then the strength and hardness will increase and the ductility decrease due to the cold working. A great deal of working can be carried out without any danger of cracking but, if this is considered to be a possibility, then copper can be softened back to its original hardness by annealing at about 550°C. Copper cannot be hardened by heat treatment (only softened). Also, it does not harden (age) with time and does not have a 'shelf life'.
5 Although copper is slightly denser than steel or cast iron, the rainwater goods and roofing sheet are much lighter since strong, stiff structures can be fabricated from thin copper sheet.
6 Due to its high ductility (ability to be extended plastically in a uniform manner), copper can easily be bent or stretched, a property which is vital in sheet when installing copper roofs and in tubes, for example when handrails are bent into curves to fit onto a staircase.
7 Malleability is the ability of copper to be compressed into very thin sections without cracking; this is important in forming fine details on interiors and embossing.
8 The low hardness values in the annealed condition (HV 40–60) contribute to good ductility and malleability.

Brass

Brass is an alloy of copper and zinc, although it may have small additions (up to 2%) of alloying elements such as lead (Pb), manganese (Mn), aluminium (Al), silicon (Si), arsenic (As) and tin (Sn) to improve its properties.

A small number of brasses have been misnamed 'bronze' because of their colouration: 'commercial bronze' (CW501L); 'manganese bronze' (CW720R); and 'architectural bronze' (CW620N). However, this misnaming does not present any difficulty in obtaining these materials since suppliers are able to deal with either method of description.

Grades

An important consideration in using brasses (or any other material) is how they may be fabricated into useful engineering structures. They may be divided according to the methods by which they may be most readily formed, as follows:

a Cold working brasses, which contain up to 35% zinc. They are very ductile and can be extensively deformed at room temperature and are widely used for the manufacture of complex components by pressing, deep drawing, spinning and other cold-forming processes. These are also known as the alpha brasses and are available as sheet, strip, rod, wire and tube. Architectural metalwork applications are shown in **Table 2**.
b Machining brasses that contain up to 3% lead which gives brasses the best machinability of any metallic alloy.
c Hot-working brasses that contain 35–40% zinc and up to 2% lead. They can be hot worked by hot stamping, rolling or extrusion and are commonly used for window frames and handrails, e.g. CW620N and CW720R, see **Table 2** and **Figure 4**.

Strength

The cold working brasses cannot be hardened by heat treatment, but during cold working their strength increases markedly to give strong, stiff, structures. The strength is retained up to about 200°C. At low temperatures, unlike mild steel, brasses retain their toughness and do not become embrittled.

Brasses may be regarded as medium-strength engineering materials with yield strengths in the range 250 to 500 N/mm^2. Included in this strength range are structural steels, some stainless steels and the higher-strength aluminium alloys.

Surface treatments

Colour range

Copper and gold are the only two metals with any distinctive colour. In brasses, the red of copper is toned to a range

Building applications	EN material number	Old UK material number	Form	Standard	Comment
Building wire	CW008A CW004A	C103 C101	Wire Wire	13601 13601	Highest electrical conductivity.
Pipework	CW024A	C106	Tube	1057	May be brazed and welded.
Plumbing fittings	CW024A	C106	Hot stampings	1254	Fittings can also be made in brass, including dezincification-resistant (DZR) brass (CW602N).
Copper roofing, cladding and flashings	CW024A	C106	Sheet 0.6 to 1.0 mm thick, width up to 1250 mm Temper R240 (1/2 hard) HV = 70–90 Tensile strength 240 N/mm^2	1172	Surface colour changes from salmon pink to chocolate brown to the stable light-green patina, which requires rainwater and takes 10 to 14 years to form in urban surroundings. Patination cannot be prevented in external applications by the use of lacquers or other coatings. Can be supplied in the pre-patinated condition.
Alloy cladding	CW502L CW503L CW450K	CZ102 CZ103 PB101 CuAl5	Sheet Sheet Sheet Sheet	1652 1652 1652 –	A range of ductile alloys that give attractive colours from golden to red to brown. All patinate to green if unprotected – colour maintained if lacquered and/or waxed.
Gutters Down pipes	CW024A CW024A	C106 C106	Rolled from sheet 0.6 mm 2.4 m lengths Rolled from sheet 0.6 mm and seam welded	1172 1172	Aesthetic appeal. Meets environmental building requirements. Easy to fit by one person, no maintenance, adds character and perceived value to a property. Cheaper than cast iron and aluminium. 100-year life.
Hopper heads	CW024A	C106	TIG welded from 0.6 mm sheet	1172	Attractive feature which collects rainwater from gutters.
Architectural metalwork – doors, windows, screens, columns, fascias	CW501L CW502L CW505L CW509L CW508L CW451K CW401J CW620N	CZ101 CZ102 CZ106 CZ109 CZ108 PB102 NS103 –	Sheet Sheet Sheet Sheet Sheet/rod Sheet Sheet Extrusions	1652 1652 1652 1652 1652/12163 1652 1652 12167	90/10 ‘commercial bronze’, one of the ‘gilding metals’. 85/15 ‘red brass’, one of the ‘gilding metals’. Most ductile of the brasses. Easily soldered. Planetarium at Greenwich clad in 8 mm welded bronze sheets. Wax finished. Nickel-silver is brass with 10% nickel. Has silver colour which in ten years of atmospheric conditions turns mottled grey-green. ‘Architectural bronze’. Aluminium imparts golden lustre with a silvery sheen. Lead gives good machinability – finished by waxing.
Handrails	CW720R CW501L CW505L	CZ136 CZ101 CZ106	Bar Rod/tube Rod/tube	12164 12163/12449 12163/12449	Manganese bronze turns a chocolate brown colour. Red to golden colour – capable of 180° bending.
Balustrades	CW501L CW505L CW620N	CZ101 CZ106 –	Tube Tube Extrusions	12449 12449 12167	As for handrails and window frames.
Restraint and load-bearing fixings Cladding (load bearing)	CW307G CC333G	CA104 AB2	Rod/extrusions Cast	12163/12167 1982	Aluminium bronzes have high strength up to 760 N/mm^2 and good corrosion resistance especially to sulphur dioxide and sodium chloride. Also resistant to stress corrosion cracking. Portcullis House (Parliament) used 6 mm aluminium bronze plate as blastproof cladding. Window frames are aluminium bronze extrusions that are three times the strength of aluminium.
Damp-roof courses, copings	CW024A	C106	Sheet	BS 5628 Part 3 2005	Copings (protecting tops of walls), damp proof courses (within walls or chimneys), weatherings (on sloping surfaces) are designed to resist rainwater. Copper is ideal for these applications since it is easy to shape, does not become brittle or corrode.
Ties, tension straps, hangers and brackets	CW307G CW451K CC333G	CA104 PB102 AB2	Sheet Rod Cast	EN 845-1	Do not need a corrosion protection system. Do not creep (deform permanently) due to long-term stress.

Table 2 Building and construction applications for copper and copper alloys

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Figure 4 Handrails

of attractive colours through to the strong yellow of CW509L (Muntz Metal) and white of nickel silvers.

Protection

Brasses will slowly tarnish in the atmosphere; outdoors, the process is more rapid due to the effect of moisture, salt (in marine environments) and pollutants such as sulphur dioxide (acid rain) in industrial areas. The tarnishing can be greatly delayed and the range of colours, which give brasses their aesthetic appeal, maintained by either lacquering or waxing (or both).

Lacquering

One of the most effective lacquers is Incralac, which is an air-drying acrylic ester lacquer containing Benzotriazole, a corrosion inhibitor. It can be relied upon to provide protection for three to eight years outdoors and for much longer indoors. Lacquers must be applied in dry, factory conditions and are not suitable for protecting components which are handled by the public, such as handrails, since acid levels of moisture in the skin (pH5.5) damages the lacquer.

Waxes

Brass may be more cheaply protected by waxing. It is important to use natural non-reactive waxes such as Carnauba (from carnauba palm tree leaves) or beeswax (from beehives), not synthetic waxes which will eventually granulate and absorb water. Natural waxes are not affected by UV light.

For extra protection it is possible to apply lacquer and wax ('belt and braces'), but in many instances waxing alone will suffice. Waxed surfaces need to be washed clean with a mild detergent and rewaxed periodically. In coastal areas subject to salt spray this may be necessary annually, whilst in urban areas the timescale is likely to be two to five years.

(Information on surface treatment with acknowledgement to Armack Chemicals.)

Bronze

Tin bronze

These are alloys of copper and tin, and were the first to be developed by ancient metallurgists about 4000 years ago. They were so important that they led to periods in time being named the Bronze Age.

Alloys with 4–8% tin are termed alpha alloys (compare with alpha brasses). However, they are harder, stronger and stiffer than the alpha brasses. Additions of small amounts of phosphorus further increase the hardness and wear resistance. They are less susceptible to stress corrosion cracking than the brasses. They have recently found applications as cladding materials, notably at the Peter Harrison Planetarium at Greenwich.

Aluminium bronze

These are alloys of copper with 5–10.5% aluminium, some having additions of iron, nickel, manganese and silicon. They are stronger than the brasses or tin bronzes with better corrosion resistance due to a hard, adherent, protective alumina film (Al_2O_3). They have an attractive golden colour, with very little tarnishing over time.

The major use for aluminium bronzes is in seawater applications, where they meet exacting Def Stan specifications (previously Naval Engineering Standards (NES)), and are widely used for Ministry of Defence (MOD) applications.

In building, aluminium bronzes have been used in:

- cladding in sheet form. The alloy CuAl5, with its golden colour, imparts a luxury appearance to any building;
- structural roof, window and wall cladding in Portcullis House, London (**Figure 5**). In this Parliament building the aluminium bronze had to be blastproof;
- buildings as ties, tension straps, hangers and brackets, where they do not need a corrosion protection system.

Figure 5 Portcullis House

In building applications, aluminium bronzes do not suffer from pitting or stress corrosion cracking.

Applications for copper and copper alloys

Typical uses for copper, brass and bronze in building and construction are shown in **Table 2**.

European and British Standards

See **Table 3**.

Pipework

Water pipework

Copper was used by the ancient Egyptians; samples taken from the pyramids are still in good condition. Copper was also used by the Romans as water pipes and cisterns. Good examples of copper in plumbing are still to be seen at the archaeological site of Herculaneum, destroyed by the eruption of Vesuvius in AD 79.

Today, copper tube is used for approximately 90% of European and North American hot and cold water supplies. Copper tube, once manufactured to BS 2871 Part 1, is now produced according to the European Standard EN 1057. Tubes are made in grade CW024A (C106) in diameters ranging from 6 to 159 mm. Copper and copper alloy fittings, once to BS 864, are now to EN 1254.

Installation of pipework for domestic water services is to BS 6700.

Over the years, the jointing techniques have developed so that, at the present time, copper tube can be joined by:

- solder – end feed and integral solder ring types;
- compression;

Standard number	Title
EN 845-1: 2003	Specification for Ancillary Components for Masonry. Ties, Tension Straps, Hangers and Brackets
EN 1057: 2006	Copper and Copper Alloys. Seamless, Round Copper Tubes for Water and Gas in Sanitary and Heating Applications
EN 1172: 1997	Copper and Copper Alloys. Sheet and Strip for Building Purposes
EN 1254: 1998	Copper and Copper Alloys. Plumbing Fittings
EN 1652: 1998	Copper and Copper Alloys. Plate, Sheet, Strip and Circles for General Purposes
EN 1982: 1999	Copper and Copper Alloys. Ingots and Castings
EN 12163: 1998	Copper and Copper Alloys. Rod for General Purposes
EN 12164: 1998	Copper and Copper Alloys. Rod for Free Machining
EN 12167: 1998	Copper and Copper Alloys. Profiles and Rectangular Bar for General Purposes
EN 12168: 1998	Copper and Copper Alloys. Hollow Rod for Free Machining Purposes
EN 12449: 1999	Copper and Copper Alloys. Seamless Round Tubes for General Purposes
EN 13601: 2002	Copper and Copper Alloy. Copper Rod, Bar and Wire for General Electrical Purposes
BS 5628-3: 2005	Code of Practice for the Use of Masonry. Materials and Components, Design and Workmanship

Table 3 Standards for copper and copper alloy products

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- push-fit fittings;
- press fittings.

In recent years, leaded solders for potable water pipework have been banned. More recently, European legislation is calling for the removal of lead-bearing materials generally.

With new requirements for site health and safety there is an increasing demand for flame-free jointing techniques and both push-fit and press fittings are suitable.

Gas pipework

As well as distributing water for domestic plumbing, copper tubes are used to safely convey natural gas to homes and businesses.

Solar water heating panels

Panels on south-facing roofs of dwellings absorb energy from the sun, which is transferred to water-filled copper pipes, typically 10 mm diameter. Copper has high thermal conductivity which is essential to provide sufficient transfer of the solar heat. The heated water is circulated by way of a pump to a copper coil inside an insulated copper tank, thus heating the water which is delivered to wherever it is needed throughout the house.

Sprinkler systems

In Britain in 2006, there were 55 800 domestic and residential fires which were responsible for at least 373 deaths and 11 200 non-fatal casualties (Fire Statistics United Kingdom, 2006, Department for Communities and Local Government). Copper domestic sprinkler installations provide protection for all habitable parts of the dwelling and are of the wet pipe type of system, i.e. one in which the pipework is constantly charged with water. Copper tube and fittings are the ideal choice for the pipework system.

BS 9251 specifies the requirements for domestic and residential fire sprinkler systems. Other building types can also be protected by copper fire sprinkler systems. Shakespeare's Globe Theatre in London (**Figure 6**) has a copper pipework sprinkler system installed, including pipework to protect the thatched roof.

Hot water cylinders

Hot water cylinders are made from copper, due to its excellent resistance to corrosion and its ductility (ability to be shaped). Copper's high thermal conductivity makes it ideal for the coil heat exchanger inside the cylinder. The cylinder itself has to be lagged to reduce heat loss.

Rainwater storage tanks

It has long been known that 'water in a copper pot does not go bad'. This is because the copper ions in tiny amounts have an antibacterial effect in sterilising rainwater, and this is beneficial when watering plants.

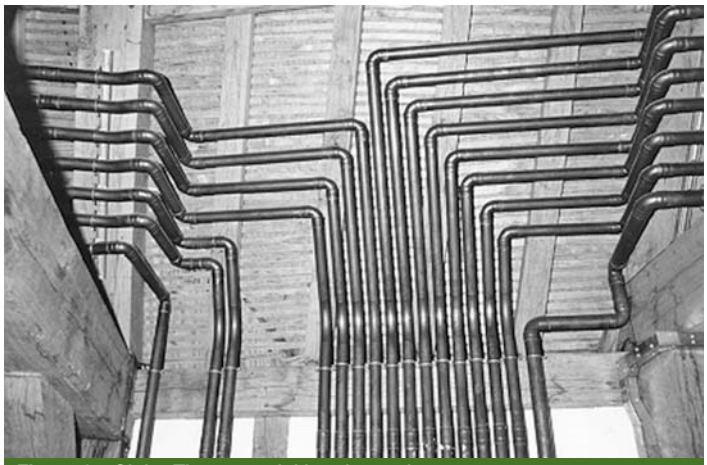

Figure 6 Globe Theatre sprinkler pipework

Architecture

Copper provides light, durable, maintenance-free structures that are naturally good looking, long lasting and fully recyclable. With a thermal expansion value of 40%, less than both zinc and lead, properly designed copper roofs minimise movements due to thermal changes, avoiding deterioration and failure. In addition, the high melting point (1083°C) of copper ensures that it will not 'creep' (flow) as some metals do.

Copper roofing can be installed using one of two methods: traditional and long strip. The traditional method involves the joining of short copper sheets, while long strip, as the name suggests, allows lengths of over 10 m to be joined. Detailing can be either with batten roll (wood block core) or with standing seams.

A correctly designed and installed copper roof will last for over 100 years. There are many examples of buildings with aged copper roofs that still perform, keeping the buildings watertight. Indeed a thirteenth-century copper church roof in Germany survives intact to this day. Copper can also be used as a cladding/façade material and there are many examples of modern buildings wrapped in copper (**Figure 7**).

During the past few years, copper alloys have been introduced as cladding materials, brass and bronze, giving architects a wider palette of colours to design with, while retaining the outstanding performance of copper. It is also now possible to use a pre-patinated copper, i.e. one that is already green in appearance (**Figure 8**).

Copper is one of very few metals that have a colour, i.e. it is not silver/grey. When used as a roof material, copper will start out the bright red colour, toning down to a brown (oxidised) and then gradually changing to the green/blue patina with which we are all familiar. It is accepted that vertical surfaces, which are not subjected to such volumes

Figure 7 Battersea Park boathouse

of rain as roofs, will patinate at a slower rate. Other factors which will affect the rate of development of the patina are the geographical location and the chemical composition of the atmosphere.

Figure 8 Brewery Square, London

Figure 9 Internal copper wall (courtesy of NDM)

Architectural metalwork

No other metals have such a range of attractive colours comparable to copper and copper alloys. The gold colour of the brasses and aluminium bronzes gives a feeling of luxury and adds prestige, both inside and outside, to any building. The red of copper, the chocolate brown of manganese bronzes and the shiny white nickel-silvers enable architects and designers to give full rein to their artistic talents. Surface treatments are available to provide additional colours if required.

The alloys are available in standard forms, are specified in European Standards and are fully recyclable. **Table 2** shows more details of building and architectural applications for copper/copper alloys.

For interior applications, designers and architects have a range of finishes and colours to choose from:

- **a** The natural salmon pink finish on bright copper sheet (**Figure 9**).
- **b** Mechanical finishes such as embossing or stamping – made possible by the high malleability of copper. Embossing imprints a pattern onto the surface, while hammering gives an irregular surface that adds texture and shadows.
- **c** Pre-patinated. The green patina formed on external copper is produced by chemicals. Sheet is supplied ready to use. The effect produced is attractive but it should be noted that the surface is difficult to clean and may be marked if handled.
- **d** Chemically toned. A very wide range of colours is available and specialist companies will provide expert guidance. Treat all chemicals with great care.
- **e** Clear coats. Applied by lacquers and/or waxes. Original colour is preserved but periodic maintenance is necessary.

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Antimicrobial

Recent research has proved that the antimicrobial nature of copper and many copper alloys is superior to stainless steels. Copper alloys are currently being evaluated in applications such as door knobs, handles (**Figure 10**), bed rails, push plates, fittings, fixtures, pens and work surfaces in hospitals to assess their effectiveness in reducing life threatening infections such as MRSA and C Difficile.

Electrical

Approximately 60% of copper produced is used for electrical applications. Copper has the highest electrical and thermal conductivity of any metal, apart from silver, leading to applications in:

- Power generation and transmission – motors, generators and transformers providing and bringing electricity safely and efficiently to homes and businesses. As newer, larger offices are built requiring air conditioning and ventilation systems, the demand for copper wire increases further.
- Electrical equipment – providing circuitry, processors and contacts for PCs, TVs and mobile phones.

Copper in transport

- Cars and lorries. The high-purity copper wire harness system carries current from the battery throughout the vehicle to equipment such as lights, central locking, on-board computers and satellite navigation systems. Electric motors, which are wound with high-conductivity wire, are used in many of these devices. The average car contains about 1 km of wire.
- Automotive radiators and heat exchangers. When good heat transfer is essential as in vehicle radiators, copper and brass are excellent choices due to their high thermal conductivity. The CuproBraze® joining process has enabled lightweight copper/brass radiators to be manufactured from thin sheet and make inroads into a market that has been dominated by aluminium. CuproBraze® heat exchangers are also being used to provide the engine cooling requirements for the latest generation of electric trains. Heat has to be removed from the electric transformers and inverters that convert the high-voltage electricity from the transmission lines into the lower-voltage, higher-amperage electricity necessary to drive the trains.

Figure 10 Copper alloy door handle

Figure 11 Sheffield Stagecoach Supertram (courtesy of Sheffield Stagecoach Supertram)

- Brake pipes (Copper Development Association, 1990). Copper and copper-nickel tubes are used for vehicle brake pipes due to their good fatigue and corrosion resistance (especially against salt on the roads), where fluid loss could be disastrous. Because of their ductility they are also easy to fit; they are standard equipment for many cars, fire engines, military vehicles, JCBs and other heavy vehicles.
- The overhead conductivity and catenary wires in electrified railways are made from a copper-silver alloy which has a good combination of high temperature strength (up to 350°C) and electrical conductivity.
- About 2% of a Boeing 747-200 jet plane is made from copper. Included in that weight is 190 000 m of copper wire.
- Electric supertrams in cities such as Manchester, Sheffield and Croydon provide clean efficient transport, being powered by four 850 V d.c. motors. The overhead contact wires are either copper-silver or copper-cadmium alloys (**Figure 11**).

Other uses of copper

- Coins. Since ancient times copper has been used in coins; the Romans used copper widely in this application. The euro coins all contain copper/copper alloys as follows:

1, 2, 5 cent pieces	Copper coated onto steel
10, 20, 50 cent pieces	Alloy of 89% copper, 5% aluminium, 5% zinc and 1% tin called 'Nordic Gold'
1 and 2 euro coins	Rings made of nickel brass and copper nickel alloys

Figure 12 Captain Cook, Greenwich

- Sculptures. Bronze has been used for centuries for statues (**Figure 12**). It is still the first choice for modern, striking works of art (**Figure 13**).
- Musical instruments. Brass instruments and cymbals.
- Cookware. Copper pots of high thermal conductivity which transmit heat evenly, resulting in food which is more evenly cooked.

Corrosion

All common metals and alloys react with a moist atmosphere and corrode. Only in hot/dry (deserts) and cold/dry environments do metals resist corrosion. However, with copper, the corrosion process is very slow; copper and bronze artefacts dating back thousands of years have been found in good condition. Copper does not rust away to nothing like mild steel. The use of copper tube in plumbing and sheet in roofing has established it as a durable metal. However, care must be taken when using copper in contact with other building materials, see bi-metallic corrosion.

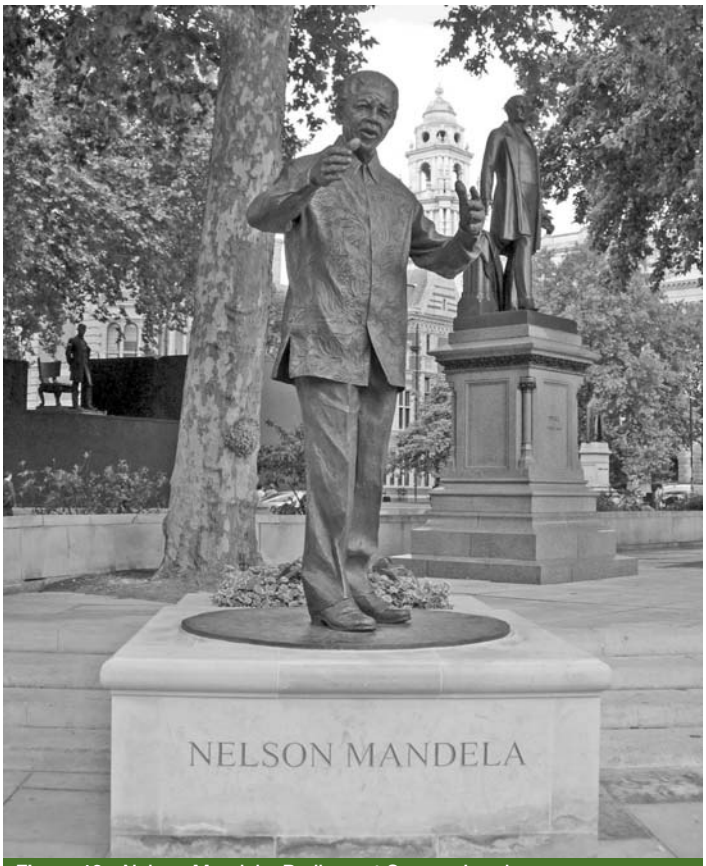

Figure 13 Nelson Mandela, Parliament Square, London

In both plumbing and architectural applications, correct design, detailing and installation will give a trouble- and maintenance-free service life.

Avoidance of pinholing

For more than 150 years, copper pipes have been used for domestic water services; the first British Standard for copper pipes was published in 1936. The continued use of the metal to this day shows that copper is a very successful plumbing material. The good corrosion resistance of copper is due to the formation of a natural, protective, stable oxide layer.

There have been, however, in a very small percentage of the millions of miles of copper pipes that have been installed worldwide, failures when conveying water, due to the formation of tiny, internal holes, called pits or pinholes. These holes result in water leakage.

The types, causes and methods of avoiding pinholing are shown in **Table 4**. It should, however, be pointed out that in some cases it may not be possible to identify precisely the exact type of pinholing.

Installation and water quality

Problems associated with incorrect design and installation can quite easily be avoided. Care should be taken to:

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Type	Cause	Avoidance measures
Erosion corrosion	A physical-chemical effect due to high water velocity – turbulent flow leading to formation of air bubbles in water which leads to removal of protective patina. Corrosion then occurs at exposed copper surface	Restrict flow rate through good design and installation practices 0.5–1.0 m/s for hot water 0.5–2.0 m/s for cold water Remove burrs from cut ends and solder beads, which increase turbulence. Filter out solids such as sand and silt. Avoid sharp bends and large changes in sections
Flux attack	Soft soldering fluxes from joining are aggressive to copper (free hydrogen chloride). Failure, if it occurs, usually takes place in first few months	Flush thoroughly after soldering in accordance with BS 6700 Use minimum amount of water-soluble fluxes
Under deposit corrosion (differential aeration)	Debris, e.g. construction material, particles brought in with unfiltered supply water, settling within pipework Differential aeration effect causes copper under deposit to become anodic and corrode. Location usually at 6 o'clock position in horizontal tubes. Time to perforation usually greater than 7 years	Avoid flow rates which are less than 0.5 m/s which allows debris to settle If supply water could contain particulate matter, especially from upland source, fit filter to incoming mains
MIC (microbially influenced corrosion)	Microbial activity, e.g. pseudomonads and/or sulfate reducing bacteria in pipework promoted by stagnant water, source of nutrients and warm water conditions (25–55°C). Hydrogen sulphide is formed due to metabolism of microbes, which causes pitting of copper (pepper-pot corrosion). Time to perforate usually greater than 7 years	Regular flushing of little-used tube runs. Filtration of supply water. Avoid stagnant flow. Lag hot water pipes (to avoid 20–60°C range). Chlorinate in larger system, such as hotels and hospitals
Carbon film pitting (Type 1)	Carbon films in tube bore resulting from extrusion greases act as effective cathodic surface. Anodic attack is concentrated at small holes in the carbon film leading to hemispherical pitting attack Occurs only in cold water pipes with hard, organically pure (borehole) water. Failure can occur within a few months	Use tubes to BS EN 1057, which have been properly cleaned. European manufacturers give a 25-year guarantee against carbon film pitting
Hot soft water pitting (Type 2)	Occurs only in soft hot water (greater than 60°C) with a low bicarbonate/sulfate ratio. Much slower than type 1 pitting and may take several years to perforate	Very rare in UK – if it occurs, either change water composition by fitting dolomitic limestone filter or reduce water temperature to below 60°C

Table 4 Avoidance of pinholing in plumbing systems (courtesy of Phillip Munn, Sheffield Testing Laboratories)

1. Flush the pipework system thoroughly with clean water on completion of any soldered or brazed joints in order to remove flux and debris.
2. Never leave a system partially filled with water for long periods immediately after installation. Failure to do this may lead to corrosion in horizontal runs, which looks like 'tidemarks', caused by differential corrosion. It is essential for new systems that fresh water is run through the system at regular intervals, promoting the development of a protective scale on the inner surface of the pipework.
3. Design out rarely used dead-legs which can lead to stagnant water. The oxygen in the water eventually becomes used up, leading to differential aeration corrosion and microbially induced corrosion (MIC).
4. Avoid excessive water velocities when selecting or setting pumps; too fast a flow can lead to erosion.

The correct water quality is equally important in avoiding problems for copper pipework.

1. In acidic water (pH less than 6) with high oxygen content (greater than 2 mg/l of oxygen) copper will dissolve to form copper ions – 'cuprosolvency'. This can lead to rapid corrosion and blue colouration of the water, so water must be treated to raise the pH level. The ideal pH of water for copper pipework should be close to 8.
2. Copper is not to be used in de-mineralised, de-ionised and distilled water. This is because ions are necessary in the water to combine with copper and form its protective patina.
3. Recent findings (Munn, 2007) indicate that phosphates in drinking water become concentrated in the copper carbonate patina, affecting its protectiveness and increasing the likelihood of 'blue water' in soft water areas and pinholing in hard water areas.

Compatibility

Galvanic series

Not all metals are compatible with copper and copper alloys. If incompatible (dissimilar) metals are in direct contact in the presence of water (the electrolyte) then galvanic, or bi-metallic, corrosion can occur. A galvanic series, see **Table 5**, which is a table of metals/alloys arranged in order of how reactive they are, is used to explain the likelihood of galvanic corrosion.

When two metals are in contact in an electrolyte it is the most anodic metal which corrodes; the further apart they are the worse the corrosion. The cathode is protected. With respect to building materials and the galvanic series, zinc (including galvanised steel), mild steel and aluminium are anodic to copper/copper alloys and will corrode. It is

Anodic end (most reactive)
Zinc and galvanised steel
Aluminium
Mild steel
Cast iron
Lead
Brass
Copper
Bronze
Austenitic stainless steel (304)
Titanium
Cathodic end (least reactive)

Table 5 A galvanic series

safe to use lead and stainless steel (304) in contact with copper/copper alloys since they are close together. The relative surface areas of the metals are a vital design consideration in determining the extent of the corrosion. A structure with a small anode area and large cathode area must be avoided.

This is important in fastener selection. The following work well with copper, with no corrosion:

- copper nails;
- brass and bronze screws and nails;
- stainless steels fasteners.

The following are anodic in relation to copper and should not be used as they will corrode:

- mild steel nails and fasteners;
- galvanised steel nails and fasteners;
- aluminium nails and fasteners.

If copper and iron/steel pipes are connected together in the presence of water, which is the electrolyte, a galvanic cell will form with the iron forming the anode and corroding. To minimise this corrosion the iron/steel pipes should be upstream of copper; if they are downstream then particles of copper will flow down the pipe forming a galvanic cell with resulting corrosion of the iron/steel.

In the case of steel tanks connected to copper pipes, galvanic corrosion is reduced by the use of insulating washers and also by the relative sizes of the exposed surfaces, in this case a large anode and small cathode.

A newly installed galvanised steel tank fed by new copper pipes suffered rapid corrosion by pitting (perforation by tiny holes). This was because copper particles which were deposited on the bottom of the tank initially formed zinc/copper galvanic cells with the consequent corroding and dissolving away locally of the zinc. This exposed the steel. Then iron/copper galvanic cells were formed, with the iron corroding locally and forming pits (pinholes) with subsequent leakage of water. This happened because the new copper pipes had not been flushed out with water to remove debris before installation.

Similar pitting occurred in newly installed kitchen steel water heaters forming perforations due to the effect of tiny particles of copper from new pipework.

Roofing materials

The bonding of copper with lead and stainless steel presents no problem.

Zinc and galvanised steel must in no circumstances be installed underneath copper in the direction of rain flow. When using copper in direct combination with galvanised steel, the materials must be isolated by the insertion of plastic foil or chlorinated rubber.

Furthermore, steel must not be mounted above copper components. This is not for fear of copper corrosion, only an aesthetic consideration; ugly rust marks will develop on the copper surface from the iron rust drips.

The bonding of copper with bare aluminium is generally considered as inadmissible. Both from the chemical point of view and aesthetic considerations, the placing of bare bright, as distinct from anodised, aluminium components under copper is not recommended. Anodised pure aluminium (with an oxide layer of 20 μm) is not normally attacked by aqueous solutions of copper. Unprotected pure aluminium will be affected to a minor extent and this from a visual aspect only. Its functional properties will not be impaired. It must not be deduced that anodised aluminium components can be installed under copper without any further thought.

To minimise risks and to ensure as long a life as possible for all materials used in construction, it is advisable to recess anodised aluminium windows and manufacture the window sills in copper. Where copper and aluminium come together, suitable inserts, such as neoprene or plastic foil, should be placed between the different metal components. The reason for this recommendation is the long-term corrosive effect of rainwater drips on projecting window sills, as well as the possibility of mechanical damage being sustained by the anodised film due to the different thermal expansion of copper and aluminium if in direct contact.

Dezincification

In certain areas, the local water supply is aggressive to conventional brass water fittings and can cause a selective leaching (dissolving) of the zinc from the brass which may eventually result in leakage or even fracture. The fittings can also be partially or completely blocked by a soft, white 'meringue' corrosion product containing basic zinc carbonate. This is easy to distinguish from limescale deposited by some hard waters which is much harder and more dense.

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Generally, hard water does not cause dezincification; it is found in certain soft water areas containing critical combinations of chloride content, temporary hardness and a high pH, usually above 8 and often above 8.3.

The solution to this problem is to use a dezincification resistant (DZR) brass CW602N (CZ132). It will be supplied heat treated in the dezincification-resistant condition and stamped 'CR'.

Information on fittings approved for dezincification resistance can be obtained from the Water Regulations Advisory Scheme, which produces publications including the *Water Regulations Guide* (replacing the *Water Byelaws Guide*) and the *Water Fittings and Material Directory*. This latter publication lists fittings and materials approved by the scheme for connection to water supplies in buildings and can be found at http://www.wras.co.uk.

Stress corrosion cracking (SCC)

Stress corrosion cracking is cracking of a metal or alloy due to a combination of stress and a specific corrosive medium. The stress may be due to:

- cold working, as in bending sheet;
- installation, as in press fits;
- service working conditions.

If either the stress or corrosive medium is absent then cracking does not occur.

The total amount of corrosion is very small but cracking occurs in a direction at right angles to that of the applied stress and may cause rapid failure. All metallic materials will suffer SCC in a unique combination for that material of stress and corrosion.

In copper alloys the cold-worked alpha brasses are susceptible to SCC in an ammonia environment. It is extremely unlikely that brass sheet or tube used in building or architectural applications would suffer from SCC since they would not be sufficiently stressed, nor encounter ammonia. However, if there is considered a possibility of cracking then a low-temperature stress-relief heat treatment at about 350°C should be carried out. This does not reduce the strength or hardness of the brass; it simply removes the internal stress.

Failure of copper pipes in plumbing systems due to SCC is also very rare, but if pipes are located close to a source of ammonia then factory produced plastic-coated copper tube should be used.

Aluminium bronzes are much superior to brasses and would not suffer from SCC.

Sustainability

Sustainability and sustainable development are very widely used terms, especially favoured by politicians keen to put a positive spin on events. The terms, however, are never defined and for this manual it is proposed to refer to the original statement (Brundtland, 1987) that sustainable development is usually considered to be: 'development that meets the needs of the present without compromising the ability of future generations to meet their own needs'.

From this definition the component parts of sustainable development have been established under the headings of:

1 environmental protection;
2 economic growth;
3 social considerations.

The sustainable credentials of copper will now be assessed under these headings.

1 Environmental protection

- Copper is a finite resource, mined from ore deposits formed in the ground millions of years ago. However, very little copper is used up, since it can be endlessly recycled without loss of properties; it is conserved for future generations.
- When copper is mined and refined to 99.9% purity, gases such as SO_2 and dust are released. These are collected by the metal producers to protect the environment. With recycling there are virtually no emissions.
- Recycling copper (and other materials) reduces landfill costs. Copper forms a very small percentage of the materials found dumped on landfill sites; it is too valuable to throw away.
- Recycling uses 15% of the energy that would be used to mine and produce the same copper. So recycling helps to conserve the world's supply of fossil fuels.
- Because of its superior conductivity, copper is the material of choice for transferring electric power. In electric motors and other components, more copper improves efficiency by reducing wasteful heat loss. This means less energy demand per unit of output which means fewer greenhouse-gas emissions that are associated with climate change.

2 Economic growth

- Copper is essential to technology, enabling peak performance from advanced microprocessors and other miniature components that drive the digital economy of today and tomorrow.
- Recycled copper is worth up to 90% of the cost of original copper. It is impossible to detect the difference between new and recycled copper, both of which are used to produce high-value, long-lasting products by companies who require no subsidies. Clearly, recycling provides an economic benefit.

3 Social considerations

- Copper is well known for its anti-bacterial properties. A strong case has been presented for the value of copper and copper alloys to help control the incidence of infection from cross-contamination due to dangerous food-borne and hospital-borne pathogens, such as *E. coli* O157, *Campylobacter*, *Listeria monocytogenes*, *Salmonella* and the difficult-to-treat Meticillin-resistant *Staphylococcus aureus* (MRSA). This case is based on

Figure 14 Offshore wind turbines (courtesy of E. O. N Renewables)

copper's intrinsic ability to quickly inactivate these dangerous microbes at both refrigerated temperature (4°C) and at room temperature (20°C).

- For centuries, copper pipes and vessels have been used to convey clean drinking water. Copper limits the growth of water-borne pathogens such as *Legionella pneumophila*.
- Copper is also a micro-nutrient vital for all forms of plant and animal life, and thus serves a critical function in agriculture to ensure soil fertility and productive yield of vital foodstuffs.
- Modern social and business life would be impossible without electricity, available instantly at the point of use. Electricity conducted by copper encounters much less resistance compared to any other commonly used metal; its electrical conductivity is 60% higher than aluminium. Copper is a key material for the generation, distribution and use of electricity and is found in cables, generators, motors and transformers. Because of its electrical conductivity, copper is a key material in energy efficiency and renewable energy generation (**Figure 14**). All household electrical appliances, electronic and telecommunication devices also contain significant quantities of copper.

Embodied energy

The embodied energy of a material (in J/kg) is a measure of the total energy consumed during every phase of the life-cycle of a product, from cradle to grave, so for copper includes energy used during mining and extraction, manufacture, disassembly and final recycling. The long life and recyclability of copper products have a positive impact on their embodied energy.

The copper industry has developed up-to-date life-cycle data for its tube, sheet and wire products. The information has been prepared in cooperation with recognised life-cycle practitioners, using international methodologies (ISO standards), leading software (GaBi), and proprietary production data collected from across the copper industry. For more information visit http://www.copper-life-cycle.org.

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Further reading

Copper Development Association publications are available online at http://www.copperinfo.co.uk.

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Chapter 43

Zinc

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Zinc is widely used in construction for corrosion protection, the die castings of components, the manufacture of brass and as sheet material for roofing and cladding. Zinc for die casting and roofing is derived from high-purity material and alloyed to tightly controlled compositions to ensure appropriate properties. For galvanising, limited low levels of impurities within zinc are permissible. Approximately 30% of zinc is currently obtained from recycled materials.

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Introduction

Zinc is the twenty-seventh most abundant element in the earth's crust and approximately 10 million tonnes are produced and used each year. Currently about 30% of the metal is from recycled sources, and this arises from re-using some 80% of the potentially recyclable products. Approximately 50% of the annual output is used for galvanising steel to prevent corrosion. In addition, 19% is used for the manufacture of brass and 16% for the production of zinc base alloys used for die casting. Zinc alloyed with copper and titanium is used as a sheet roofing and cladding material. Other end uses of zinc include batteries and in zinc compounds, such as zinc oxide used in rubber tyre production; also a range of zinc derivatives are used in pharmaceutical, cosmetic and dietary products. Overall, approximately 45% of the annual production is used within the construction industry.

Resources

Zinc is mined in over 50 countries throughout the world, but the main producers are China, Australia and Canada, followed by the USA, Peru, Mexico and Kazakhstan. Total world production is rising annually and, at the current rate of extraction, Australia, with the largest economic demonstrated resource, has some 20 years of known production available. The majority of mines (80%) are underground; however, open pit mines account for a significant proportion of world production.

Zinc deposits vary significantly in composition, but the most common ore, which accounts for 95% of world production, is sphalerite (ZnS) or zinc blende. The zinc sulphide ores occur in association with other metals, particularly lead and silver but also copper and gold, which are separated out during the refining process.

Production

At the mine, the zinc ore is crushed and concentrated by flotation techniques to separate the metal-bearing minerals from waste, increasing the proportion of zinc mineral from 5–15% to about 55%. The concentrate is then transported to the refinery for processing. First, the ore is roasted to convert the zinc sulphide into zinc oxide, releasing sulphur which is captured as sulphur dioxide and converted into sulphuric acid as a commercial by-product. The crude zinc oxide is then converted into zinc metal by either an electrolytic or thermal processes.

In the electrolytic process, which accounts for approximately 90% of production, the zinc oxide is dissolved in sulphuric acid, leaving other metals precipitated or undissolved. The solution is then electrolysed between lead anodes and aluminium cathodes, causing virtually pure zinc to be deposited on the aluminium. The deposited zinc is removed and cast into ingots of 99.95% purity.

In the thermal process, which is used for concentrates with a high lead content, the crude zinc oxide is charged with coke into an Imperial Smelting Process furnace. The carbon monoxide, produced at over 1000°C, reduces the zinc and lead oxides to metals. The lead, containing any copper and precious metals, runs to the base of the furnace, whilst the zinc vapourises and is captured by a molten lead spray. When the molten lead and zinc mix is subsequently cooled, the zinc separates out and is cast into ingots, while the lead is recirculated back to the condenser. Only a small proportion of zinc is produced worldwide by this thermal process as it is energy intensive and the zinc contains some 2.5% lead which may require further refining.

Properties

Zinc is a blue-grey metal which rapidly tarnishes in air with the production of a grey patina of basic zinc carbonate. The

Density	7.14 gcm^{-3}
Melting point	419°C
Boiling point	907°C
Linear coefficient of expansion (25°C)	30.2 μm.m^{-1}K^{-1}
Thermal conductivity (18°C)	113 Wm^{-1}K^{-1}
Mohs hardness	2.5
Modulus of elasticity	70 GPa
Electrical resistivity (20°C)	5.9 μΩm
Standard electrode potential (against hydrogen electrode)	−0.76 V

Table 1 Physical properties of -zinc

metal is hard at ambient temperatures and brittle when cold, and should not be worked at temperatures below 10°C. Zinc melts at 419°C and boils at 907°C and reacts with both acids and alkalis (**Table 1**).

Grades and alloys of zinc

Zinc is available as high grade (HG) 99.95% and special high grade (SHG) 99.99% purity. The metal is used extensively for the corrosion protection of steel by various forms of application. The pure metal is also used as anodes for the cathodic protection of steelwork. Zinc is alloyed with copper to form brass, with aluminium, magnesium and copper for die casting, and with copper and titanium for sheet roofing and cladding. The purity of primary zinc is defined by chemical composition in the standard BS EN 1179: 2003, with colour coding to assist identification (**Table 2**). The standard limits the individual and overall content of specific metallic impurities. High-purity grades are required for die casting. Continuous galvanising grade (CGG) contains some aluminium and lead.

Secondary zinc, that is material obtained by recycling zinc sheet and scrap or that which is recovered from process residues, is covered by BS EN 13283: 2002 (**Table 3**). These grades may be used for hot-dip galvanising and for the production of brass.

Zinc coatings

Zinc coatings offer good corrosion resistance to iron and steel within most environments, largely due to the electrochemical difference between the metals. The two metals in contact form an electrical cell in which the zinc acts as the anode dissolving preferentially, thus preventing the rusting of the iron or steel. Protection is therefore afforded, even where there are discontinuities in the zinc-rich coating due to cut edges or local damage. In addition, zinc alone corrodes more slowly in the atmosphere than steel. A range of different methods are available to apply the zinc coatings to the substrate; however, hot-dip galvanising is the most commonly used for bulk materials, although some components may be flame sprayed. Smaller units are usually electroplated or sherardised and painting with zinc-rich paint is an alternative. In addition to galvanising with zinc, metallic coatings may be zinc–iron alloy (ZF), zinc–aluminium alloy (ZA) or aluminium–zinc alloy (AZ).

Grade classification	Nominal zinc content	Colour code
Z1	99.995	White
Z2	99.99	Yellow
Z3	99.95	Green
Z4	99.5	Blue
Z5	98.5	Black

Table 2 Classification and colour coding of primary zinc by composition

Grade classification	Nominal zinc content	Typical sources
ZSA	98.5	Process residues
ZS1	98.0	Recycling of scrap
ZS2	97.5	Recycling of scrap

Table 3 Classification and typical sources of secondary zinc by composition

Hot-dip galvanising

Hot-dip galvanising may be a batch or continuous process. In either case the steel must be chemically cleaned to remove grease, rust and mill scale before application of the molten zinc. In the batch process, any grease is removed first by dipping into a degreasing agent, followed by the dilute hydrochloric acid removal of rust and mill scale, then rinsing. Components are finally dipped into a flux solution to remove the last traces of oxide, leaving a clean surface and ensuring full surface wetting by the molten zinc. On immersion in molten zinc at approximately 450°C, a series of iron–zinc alloy layers is rapidly formed at the surface of the steel. These surface reactions slow down and for most components are sufficiently complete after immersion for 4 or 5 minutes. The outer layer of molten zinc then crystallises as the component is removed from the zinc bath, leaving a zinc-rich layer of typically 85 μm in thickness for steel of 6 mm or more in section.

For the continuous galvanising of steel sheet, the surfaces are cleaned and the strip is run through a bath of molten zinc and then blasted with a jet of compressed air to remove the excess metal. The thickness of the zinc layer is controlled by the immersion time, bath temperature and the wiping process. Most continuous steel strip receives an average coating thickness on both faces of 20 μm. The coating is measured either as mass, e.g. 180 gm/m^2 (total on both sides), or thickness, e.g. 14 μm (on each face), and these are coded, e.g. G60.

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Most continuously galvanised sheet steel is now coated with an aluminium alloy of zinc, containing approximately 5% aluminium (ZA), as this confers improved end-product performance. However, for significantly enhanced corrosion resistance, the 55% aluminium/43% zinc/1.6% silicon alloy is used. This aluminium-rich alloy (AZ) typically offers a three-fold enhanced corrosion resistance over standard zinc galvanised steel of the same coating thickness. The aluminium-rich process sets up a complex surface microstructure consisting of an aluminium-rich phase and a zinc-rich phase with an intermediate bonding layer of an aluminum/iron/zinc/silicon alloy. The alloy has a metallic lustre which varies with production conditions.

Hot-dip zinc–iron coating (ZF) is produced by standard immersion of the iron or steel product in molten 99% zinc, followed immediately by annealing which generates an iron content in the coating normally within the range 8% to 12% iron. Annealed (ZF) coatings have a uniform dull matt grey appearance.

The appearance of standard zinc galvanised steel may be the normal spangle or crystalline finish (N). Alternatively a minimised spangle finish (M) may be specified if the crystalline finish would affect subsequent painting. The efficiency of the coating is not affected by the crystallinity of the finish.

The standards covering continuous hot-dip galvanised sheet steel are BS ISO 3575: 2005, BS ISO 4998: 2005, BS EN 10143: 2006 and BS EN 1034: 2009. The Standard BS EN 505: 2000 refers to fully supported roofing products of steel sheet, BS 7371-6: 1998 refers to metal fasteners, BS EN ISO 1461: 1999 refers to coatings on fabricated articles, and BS ISO 14788: 2005 refers specifically to zinc–5% aluminium alloy coated sheet steel.

Steel wire may be coated with zinc or zinc aluminium alloy (ZA) either by the hot-dip process or in electrolytic solution to BS ISO 7989-2: 2007 and BS EN 10244-2: 2001.

Thermal spraying

Thermal sprayed zinc coating is produced by heating zinc wire to melting point in a flame or arc-activated gun which projects the molten zinc in a stream of inert gas onto the prepared surface. The surface of the metal to be coated must be cleaned by abrasive blasting to a uniform texture, specified in BS EN ISO 8501-1: 2007 as grade Sa3, and thermally sprayed normally within 4 h to prevent any oxidation. Either 99.99% zinc or a zinc/aluminium 15% alloy may be used within the process to BS EN ISO 2063: 2005. Coating thicknesses usually range between 100 μm and 200 μm depending upon client requirements. The zinc coating is porous and requires sealing before any moisture is absorbed. Thermal spraying is often used for components which are too large to be dipped in a galvanising bath.

Electroplating

The electroplating of small components, which require a fine smooth finish, is carried out by electrolytic deposition from a zinc salt solution. The system is typically used for threaded components and produces thicknesses normally from 3 μm to 12 μm or up to 30 μm where required. These thin finishes are not suitable for extended outdoor exposure. The specification for electroplated zinc is covered by BS 7371-3: 1993.

Mechanically deposited zinc coating

Sherardising involves the coating of small components by tumbling within a mixture of zinc powder and an inert material such as sand at temperatures just below the melting point of zinc. The components require cleaning prior to the solid diffusion process which produces film thicknesses between 15 μm and 30 μm as standard, although 45 μm can be achieved. Sherardising is covered by the Standards BS 4921: 1988, BS 7371-8: 1998 and BS EN 13811: 2003. An even grey matt finish is obtained.

In an alternative process, zinc coatings may be mechanically deposited at ambient temperature by tumbling small components within a solution containing zinc powder and glass beads. The process is covered by the Standard BS EN ISO 12683: 2004.

In zinc flake coating to BS EN 13858: 2006, small components are immersed in zinc flake liquid, which may also contain a small proportion of aluminium flakes. The components are then spun to remove excess liquid, and the material is heated to cure the organic or inorganic liquid matrix. The use of metal flakes rather than powder ensures that the zinc lies parallel to the substrate ensuring good continuous coverage.

Zinc-rich paints

Zinc-rich paints contain very high levels of zinc dust, and may be applied to clean and scale-free steel surfaces by brushing or spraying. The dried film, containing a high proportion of zinc, acts as a physical barrier to the environment and also gives cathodic protection to the substrate. Zinc-rich paints are classified either as inorganic or organic according to the binder. Generally, inorganic alkali–silicate zinc-rich paints contain over 77% of zinc in the dried coating and are usually applied by spraying. The organic paints mostly based on epoxy binders contain approximately 90% zinc in the finished layer and may be sprayed or brushed. The Standard BS EN ISO 12944-5: 2007 defines zinc-rich primers as having a minimum 80% by mass of zinc of non-volatile material.

Durability of zinc coatings

The corrosion protection afforded by zinc coatings depends upon the thickness of the coating and the environmental

Zinc coating	Typical coating thickness: μm	Anticipated coating life: years
Electro-galvanised	5	3
Continuous hot-dip galvanised	20	13
Sherardised	30	20
Batch hot-dip galvanised	85	57

Table 4 Anticipated durability of zinc coatings

conditions (**Table 4**). Local environmental conditions in the UK can be determined from the Galvanisers' Association Zinc Millennium Map which indicates that typical corrosion rates of 1.5 μm per year can be anticipated for most UK regions. This corresponds to the medium corrosion risk associated with urban inland and mild coastal environments in BS EN ISO 14713: 1999, which gives an annual thickness loss of between 0.7 and 2 μm for these situations.

Under normal exposure conditions, new galvanised zinc will tarnish in air due to the formation of a patina of basic zinc carbonate. This film then protects the underlying zinc from further oxidation. If, however, the galvanised material remains damp and within limited air circulation, the zinc will be converted into a white deposit of zinc oxide or hydroxide, which in mild cases can be lightly brushed off, but in more serious cases may have seriously affected the service life of the component. Where extensive staining has occurred, possibly with signs of rusting, remedial treatment on the coating is necessary, particularly on components with thin zinc coatings. Where the risk of wet storage staining remains, the surface of the component may be protected by repassivating the galvanised surface with a solution of sodium dichromate in dilute sulphuric acid. Under most atmospheric conditions the corrosion rate of the zinc layer is slow. However, the presence of atmospheric sulphur dioxide, and to a lesser extent contact with chlorides and long periods of wetness, increases the corrosion rate.

The effectiveness of the zinc coating depends upon its tenacious metallurgical bond to the steel. Continuously hot-dip steel sheet can be mechanically formed without affecting the galvanised layer as the coating has a very thin zinc–iron alloy layer. However, batch-dipped components should not be worked after coating as this may cause cracking or flaking of the thicker hard zinc–iron alloy layer, although the surface is protected against impact damage by the softer outer layer of pure zinc.

Die castings

Zinc die casting is used for the manufacture of a wide range of components. Most zinc die castings are now produced by high-pressure die casting which ensures a near net-shape casting with a good surface finish. Dimensional accuracy to close tolerances can be achieved for intricate shapes including thin wall components which then require little or no subsequent machining. Zinc castings have good ductility, high impact strength and are easily plated. Some air is trapped within the zinc metal during casting, causing porosity, but with new techniques this is limited to around 3%. However, castings with porosity when subjected to temperatures over 100°C may blister due to the expansion of entrapped air.

Special high-grade zinc of over 99.99% purity is used for die casting as low levels of impurities lead to embrittlement. The Standard BS EN 12844: 1999 specifies the quality of the zinc or zinc alloy used and specifically excludes the use of recycled materials which could cause contamination. In particular the levels of lead and cadmium must be within the specified maxima to BS EN 1774: 1998. Alloys are designated by symbol or by number for foundry purposes, but may be described by a short designation and/or colour code for marking (**Table 5**).

Alloy symbol	Alloy number	Short designation	Colour code	Composition %				
				Al	Cu	Mg	Pb: max	Cd: max
ZnAl4Cu3	ZL0430	ZL2	White/green	3.8–4.2	2.7–3.3	0.035–0.06	0.003	0.003
ZnAl4	ZL0400	ZL3	White/yellow	3.8–4.2	0.03 (max)	0.035–0.06	0.003	0.003
ZnAl4Cu1	ZL0410	ZL5	White/black	3.8–4.2	0.7–1.1	0.035–0.06	0.003	0.003
ZnAl8Cu1	ZL0810	ZL8	White/blue	8.2–8.8	0.9–1.3	0.02–0.03	0.005	0.005
ZnAl11Cu1	ZL1110	ZL12	White/orange	10.8–11.5	0.5–1.2	0.02–0.03	0.005	0.005
ZnCu1CrTi (includes chromium and titanium)	ZL0010	ZL16	White/brown	0.01–0.04	1.0–1.5	0.02 (max)	0.005	0.004
ZnAl27Cu2	ZL2720	ZL27	White/violet	25.5–28.0	2.0–2.5	0.012–0.02	0.005	0.005

Table 5 Composition and designation system for a selection of zinc alloys to BS EN 1774: 1998

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Example: Designation of zinc alloyed with 4% aluminium and 1% copper

Designation by symbol ZnAl4Cu1
zinc with 4% aluminium and 1% copper

Designation by number ZL0410
ZL for zinc alloy, 04% aluminium, 1% copper, 0% for any other alloying element if over 1%

Short designation and colour code ZL5 white/black

The most commonly used alloys for pressure die casting are ZL3 and ZL5. These have ultimate tensile strengths at room temperature of 280 and 330 MPa respectively, when subjected to rapid solidification in die casting processes. Elongation to failure is typically 6% with an Izod impact value of over 50 J at room temperature. Initial changes in volume can be accelerated by ageing at 100°C for 3–6 h. Creep resistance of ZL3 and ZL5 is low, but greater in ZL8, also in ZL16 which is alloyed with copper, chromium and titanium.

Zinc die casting is used for the manufacture of many small components used within the construction industry including door and window hardware, bathroom fittings and security locks. As zinc and its alloys tarnish, most products are coated either by dipping or electroplating. For components such as sink faucets, the zinc casting is usually electroplated with copper, then nickel and finally chromium to ensure durability and a highly polished finish.

Traditional sand or gravity die casting, using zinc alloys ZL8, ZL12 and ZL27, offers an alternative to aluminium, cast iron and brass for a range of applications where strength, hardness and wear resistance are required. Finishing by machining may be required, but is easy to perform. Typical applications include electrical fittings and hardware components.

Wrought zinc

Wrought zinc and zinc alloys are produced as rolled sheet, strip, and foil, extruded sections, also drawn rod and wire.

High-purity zinc (99.995%) alloyed with a minimum of 0.06% titanium and 0.08% copper to BS EN 988: 1997 is used as sheet roofing and cladding material (**Table 6**). The titanium increases creep resistance and the copper improves mechanical strength. In addition the titanium/copper alloy has a lower coefficient of expansion at 22 $\mu m.m^{-1}K^{-1}$, which is advantageous in relation to movement joints.

Nominal thickness: mm	0.6	0.65	0.7	0.8	1.0	1.2	1.5
Nominal weight: kg/m^2	4.3	4.7	5.0	5.8	7.2	8.6	10.8

The preferred sheet lengths are 2 m and 3 m, although 1 m and 2.5 m are available. Sheet thicknesses of 0.3 mm to 2 mm are also available.

Table 6 Titanium zinc alloy preferred sheet thicknesses (BS EN 988: 1997) and associated weights

Figure 1 The Centre for Mathematical Studies at Cambridge University. Architect: Edward Cullinan Architects (photograph by Arthur Lyons)

This permits the construction of roof bays up to 10 m, or in certain cases up to 16 m in length, depending upon design considerations including bay width. In long bay construction, a section of 1.3 m is fixed, while the remaining length is secured with sliding clips permitting thermal movement. The position of the fixed section depends upon the roof pitch.

The alloy sheet to BS EN 988: 1997 has a minimum tensile strength of 150 MPa and a minimum of 35% elongation at fracture. The material may be pre-patinated to a light, dark or slate grey if required, and very soft colours – blue, red and green – have recently become available. Also, heat-treated factory-applied laquer finishes give a wider range of colour options. In addition to roofing and cladding, titanium zinc is used in construction for flashings and rainwater goods.

The Centre for Mathematical Studies at Cambridge University, designed by Edward Cullinan Architects (**Figure 1**), is an excellent example of zinc alloy roofing. It illustrates the use of long bays and shows the blue-grey patina associated with the durable titanium alloy.

The Code of Practice CP 143-5: 1964 for zinc sheet roof and wall coverings which illustrates traditional roll cap covering with welted joints or drips, was reconfirmed in December 2007. Self-supporting metal sheet roofing and cladding are covered by the Standards BS EN 506: 2008 and BS EN 14782: 2006, while fully supported metal sheet roofing and cladding are covered by the Standard BS EN 14783: 2006.

Within construction, metallic contact between zinc and copper, brass, iron or unpatinated lead should be avoided. Contact with bitumen and acidic timbers such as oak,

chestnut, larch, cedar, pine or the soluble salts from cement or plaster can cause corrosion.

Zinc aluminium alloy with an aluminium content between 18% and 30% can be superplastically deformed. The material is initially subjected to a multi-stage heat treatment to produce a uniform fine grain structure. The superplastic zinc (SPZ), typically with a 22% aluminium composition, is then deformed at constant speed followed by a dwell period under constant load at a temperature of approximately 260°C. The repeated operation is slow, requiring the metal to flow into the required mould form as the two halves of the die are gradually moved together. Superplastically deformed products should not be used at temperatures above 100°C, unless they are heat treated to eliminate their superplasticity.

Cathodic protection

Cathodic protection is covered by the Standard BS 7361-1: 1991. Zinc anodes cast from high-purity zinc, containing less than 0.0014% iron, perform satisfactorily as protection for steelwork and pipes in contact with salt water or soil. A good electrical connection is required between the sacrificial anode and the structure requiring protection. High-purity zinc or appropriate alloys are required to ensure that the zinc surface remains active.

Soft solders

A wide range of soft solders is covered in the Standard BS EN ISO 9453: 2006, including tin-zinc with and without the addition of bismuth. Compositions of typically 90% tin with zinc or with zinc and bismuth melt in the range 190°C to 199°C.

Sustainability and recycling of zinc

Currently 70% of current zinc production originates from mined ores, while 30% arises from recycled material. Of the zinc available for recycling, approximately 80% is now captured and reused. Production off-cuts, roofing sheets etc. are easily recycled, but also the zinc on recycled scrap galvanised steel is recovered within the Electric Arc Furnace used for recycling the steel. The zinc rapidly vaporises and is collected from the gas stream as dust which is then enriched, oxidised to zinc oxide then converted back to the zinc metal. Primary production of zinc from raw materials consumes approximately 3600 kW per tonne of metal.

Brass is one of the most efficiently recycled materials. Brass components rarely go to landfill sites but are virtually always recovered at waste collection centres.

Industrial activity over the last century led to increased atmospheric levels of zinc, but since the 1970s, environmental awareness has led to the progressive reduction in zinc emissions both to the atmosphere and to industrial water outflows. Additionally as sulphur dioxide in the atmosphere is reduced, the corrosion rate of zinc products releasing material into the air is also reduced.

Zinc as a trace element

Zinc is an essential trace element for all living organisms. Most of the requirement for human health is obtained from meat, particularly red meat, but also from poultry, fish, seafood, cereals and dairy products. Zinc deficiency can lead to health problems, equally an excess can cause adverse reactions. Crops require traces of zinc for normal growth and deficient soils can be appropriately treated with fertilisers.

Further information

The British and European Standards are constantly being updated, and specifiers should ensure that the details of the appropriate current versions are observed. Draft European and British Standards are available for review and comment from the British Standards Institution before their adoption across the European Union or within the UK respectively. Older texts often refer to standards that have been either completely withdrawn or partially replaced by new standards.

The bibliography lists some standard texts on zinc but also a selection of advisory bodies and commercial organisations relating to specific parts of the industry, such as galvanising, die casting and sheet metal roofing. Brass is not included in this text as it is included in sections relating to copper. A selection of texts relating to the business aspects of zinc production and international consumption are also included for wider commercial interest.

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Chapter 44

Titanium

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Titanium and its alloys are important engineering materials widely used for their excellent corrosion resistance and high strength combined with a density two-thirds of that of steel. Used extensively in the aerospace, chemical engineering, and oil and gas sectors, titanium alloys can also be used in architectural and structural applications.

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Sources and production

Titanium metal is derived from rutile (TiO_2) or ilmenite ($FeTiO_3$) ores, with rutile being preferred due to the low level of iron content. As can be seen in **Table 1**, the main sources for both rutile and ilmenite are Australia, South Africa, Canada and China. Mine production in 2006 was 5% greater than in 2005.

Elemental titanium was discovered in 1791, but the extraction of metallic titanium did not occur until 1910 when M. A. Hunter, an American chemist, reduced titanium chloride ($TiCl_4$) with sodium at temperatures between 700°C and 800°C. This formed the basis of the Hunter Process. Full commercialisation of titanium was a result of the development of the Kroll process in 1946, which reduced $TiCl_4$ with magnesium. The Kroll process is today the predominant method for the production of titanium sponge from titanium ores.

Kroll process

The hazardous nature of the Kroll process, slow reaction times and low production volumes account for a large part of the high cost of titanium.

In the Kroll process, rutile is reacted with coke and chlorine gas at a temperature of 1000°C to produce $TiCl_4$. The $TiCl_4$ is purified by continuous fractional distillation and the recovered chloride is reprocessed and fed back into the system. The $TiCl_4$ is then reacted with magnesium at 800–900°C in an argon-filled pressure vessel over a period of three days, with magnesium chloride periodically tapped off. The resulting batch of porous titanium sponge (**Figure 1**) is purified by leaching or heated vacuum distillation prior to compacting into an electrode and melting in a vacuum arc remelting (VAR) furnace (**Figure 2**) to produce titanium ingot (**Figure 3**). As of 2007, worldwide production of titanium sponge is about 110 000 metric tons per annum, with estimates that it will exceed 150 000 metric tons by 2010. The main sources of sponge are Russia, Ukraine, Japan and the USA.

VAR ingots are typically 3–6 t in weight and 650–900 mm in diameter. In addition to titanium sponge, titanium scrap can be remelted to render the production of ingot more economical.

An alternative to the VAR of titanium is plasma arc melting (PAM) or electron beam cold hearth melting (EBCHM). These processes were developed to prevent the formation of certain types of inclusions or microstructural segregation that can occur in VAR and that can have a detrimental effect on the mechanical properties of titanium alloys, particularly fatigue strength. **Figure 4** illustrates the basic concept: titanium scrap or sponge is fed into a water-cooled copper hearth and is melted using a plasma arc (PAM) or electron beam (EBCHM). A solid layer (skull) of titanium forms on the copper hearth. During melting, high-density inclusions sink and become trapped in the solid 'skull' of material at the bottom of the hearth while low density inclusions float and can be removed by interaction with the heat source. Neutral-density inclusions dissolve in the molten material. The molten material is then fed into a second water-cooled copper crucible, and gradually solidifies to produce a continuously cast slab, as in **Figure 5**. Slabs vary in size according to customer specification, typically up to 66 cm thick, 137 cm wide and 4.2 m long.

Primary processing

As with other metals, titanium ingot has to be processed into bar, plate, sheet, tube or wire to be of use to the end user. **Figure 6** illustrates a typical layout for processing of titanium ingot. Final mechanical properties result from very careful control of the thermomechanical processing regimes to produce the correct microstructure for the alloy in question.

Secondary processing

Secondary processing of titanium alloys includes all the traditional metal-forming techniques, including casting, forging, pressing, welding, drawing and machining.

	Mine production: millions of tons		Reserve base
	2005	2006	
Ilmenite			
Australia	118	121	16 000
Brazil	13	13	1 200
Canada	73.1	78	3 600
China	45	47.5	35 000
India	29.7	29.7	21 000
Mozambique	–	–	2 100
Norway	38.1	38.1	6 000
South Africa	86.7	89.3	22 000
Ukraine	21.8	27.3	1 300
United States	30	30	5 900
Vietnam	9.5	6.4	750
Other countries	13.6	14.4	7 800
World total (ilmenite, rounded)	**480**	**500**	**120 000**
Rutile			
Australia	16.3	17.1	3 100
Brazil	0.3	0.3	350
India	1.8	2	2 000
Mozambique	–	–	57
Sierra Leone	–	8	360
South Africa	10.5	10.8	2 400
Ukraine	5.7	6.2	250
United States	–(1)	–(1)	180
Other countries	–	–	1 700
World total (rutile, rounded)	35.1	44.4	10 000
World total (ilmenite, and rutile, rounded)	**520**	**540**	**130 000**

Source: US Geological Survey (2007).
Note: (1) US rutile production is included with the ilmenite to avoid disclosing company proprietary data.

Table 1 World mine production of ilmenite and rutile and reserve base

Figure 1 Impurities being removed from titanium sponge (courtesy of A. T. I. Wah Chang)

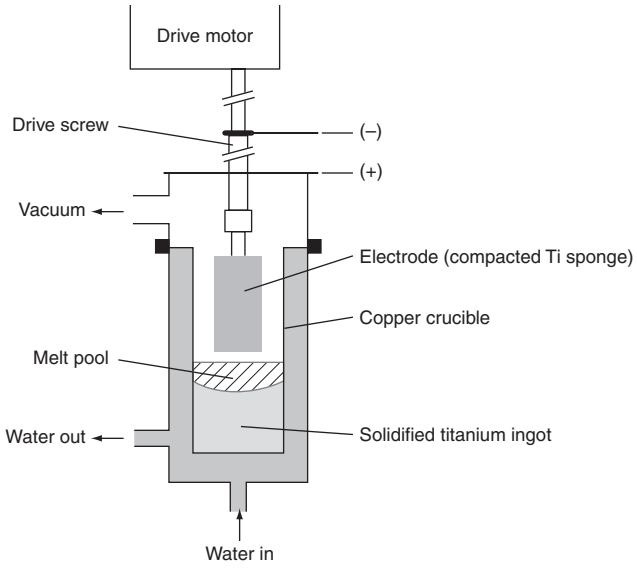

Figure 2 Vacuum arc remelting of titanium sponge

Titanium's rate sensitivity of flow stress makes it ideal for superplastic forming and new techniques to produce and consolidate titanium powder are being investigated. Special consideration has to be taken for titanium's affinity for oxygen during welding and its work-hardening characteristics during forming. The Young's modulus of titanium alloys is typically half of that of steel, which results in a greater spring-back when cold forming.

Figure 3 Titanium ingot (courtesy of VSMPO-Tirus)

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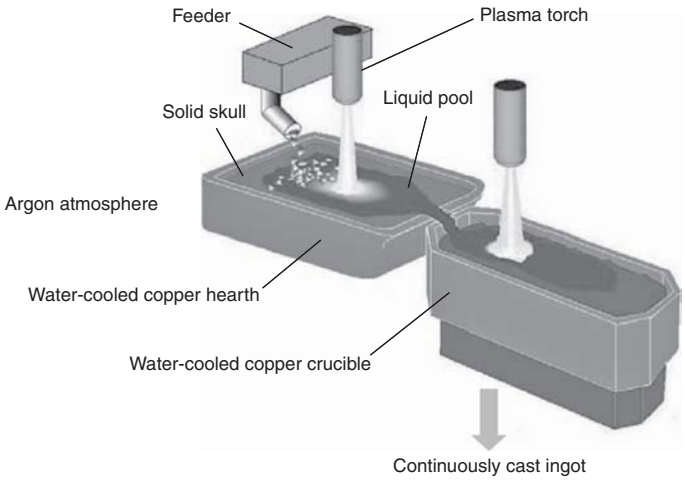

Figure 4 Production of titanium ingots for use in the aircraft industry via PAM

Figure 5 EBCHM slab (courtesy of ANTARES)

Titanium alloy categorisation

Pure titanium is allotropic, such that below the beta transus temperature of 883°C it has a close-packed hexagonal microstructure (alpha phase) but a body-centred cubic microstructure (beta phase) above 883°C until the melting point of 1678°C (**Figure 7**). The addition of alpha stabilisers such as Al, Sn, Zr, O, N or C stabilise the alpha phase and cause the phase change to occur at a higher temperature; conversely, beta stabilisers such as Mn, V, Nb, Ta, Cr, Mo or Fe stabilise the beta phase and cause it to occur at lower temperatures. In practice, titanium alloys may contain both alpha and beta stabilisers to create a microstructure with the desired mechanical properties, so titanium alloys can therefore be categorised based on the predominance of either the alpha phase or the beta phase.

Several international standards also exist that specify the mechanical properties of titanium, their cleaning, and heat treatment, the most common of which is the ASTM standard by which the different alloys are known. Standards exist also for specific product forms, such as welded pipe or sheet, or for specific uses, such as in medical applications. Information on the parameters for the thermomechanical processing of titanium alloys to achieve the desired properties is often proprietary. In addition to the alloys listed in the ASTM standards, there are many proprietary grades which are not listed or have gone out of production.

When categorising titanium by microstructure, there are four broad categories, each alloy having properties that make it more suitable than others in particular applications.

Commercially pure grades

Commercially pure grades contain no alloying additions and therefore contain only the alpha phase. They are the softest of the alloys available and have the highest ductility and formability. Their strength can be increased significantly only by work hardening. The four commercially pure grades are grades 1–4, with increasing oxygen content from grade 1 to grade 4 resulting in higher strength but lower ductility. They have excellent corrosion resistance, which can be further enhanced by the addition of palladium or ruthenium, which results in ASTM grades 7, 11, 16, 17, 26 and 27. Grade 2 is approved in NACE MR-01-75 standard for use in sour service conditions.

Near alpha and alpha alloys

These are alloys with a predominantly alpha phase and which often have very good creep properties and higher strength than the commercially pure grades, achieved by alloying with aluminium vanadium, tin, molybdenum or nickel. These include grades 6, 9, 12, 18 and 28. Grades 12 and 28 are approved in the NACE MR-01-75 standard for use in sour service conditions, while grade Ti-5-2.5

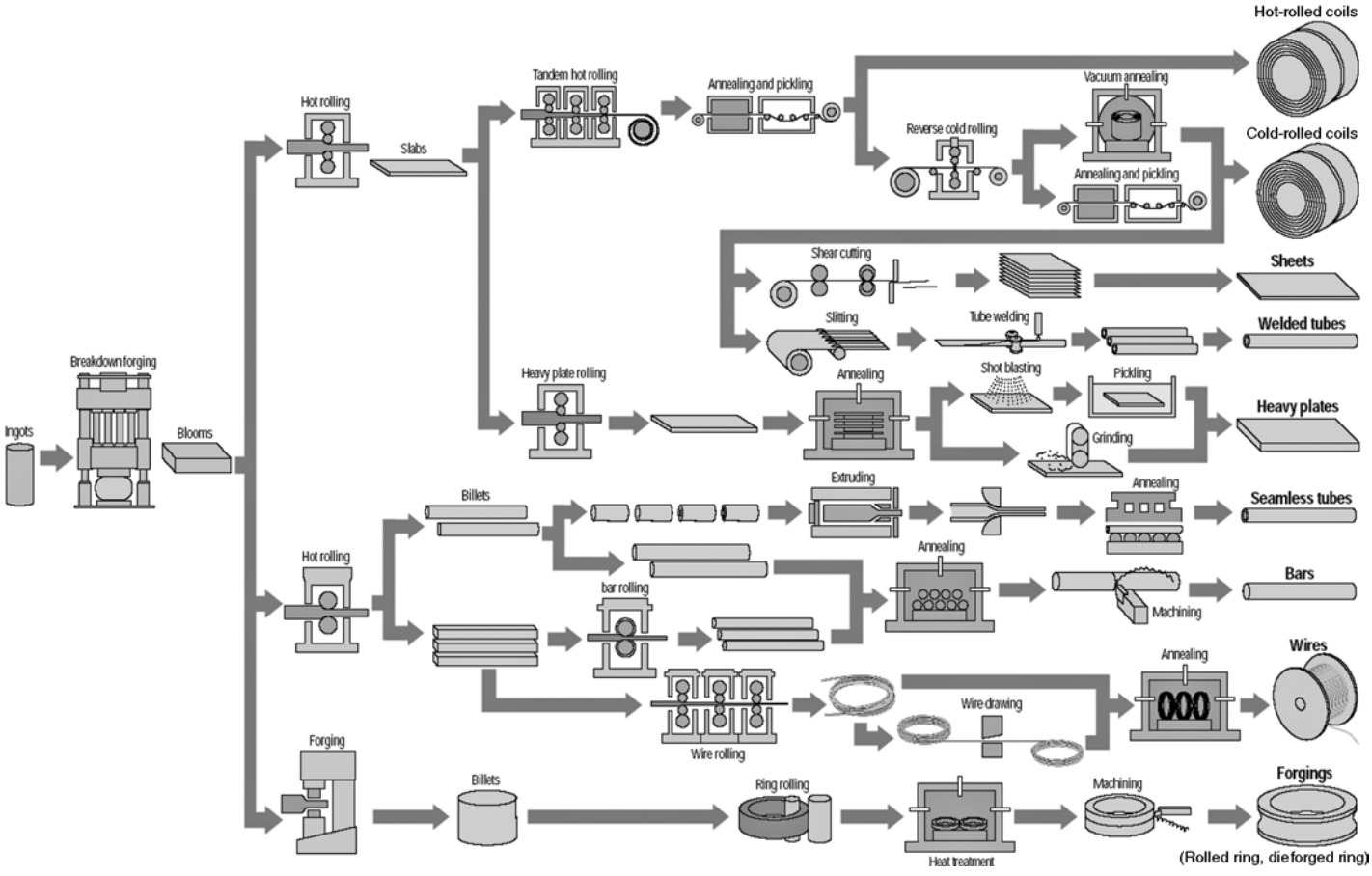

Figure 6 Typical processing of titanium ingot

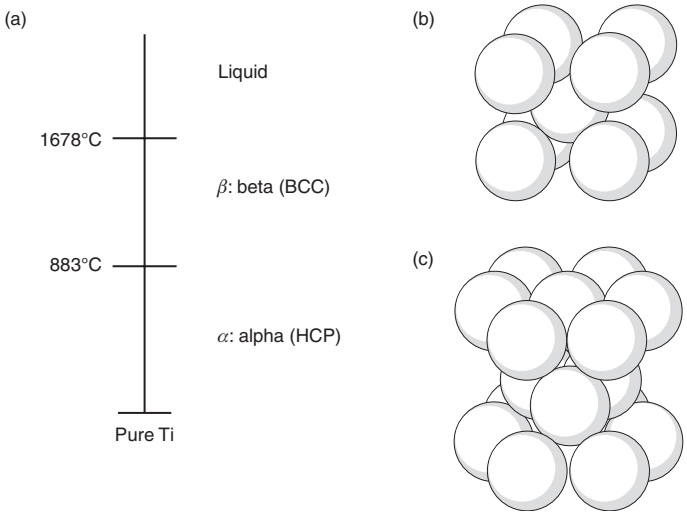

Figure 7 Phases in titanium: (a) beta transus temperature; (b) body-centred cubic structure; (c) close-packed hexagonal structure

ELI has excellent cryogenic properties allowing it to be used as low as −255°C.

Alpha-beta alloys

Alpha-beta alloys have a mixture of the alpha and beta phase and are medium to high strength. They can be heat treated to higher strength and this category includes the workhorse of the titanium industry, Ti 6Al-4V, ASTM grade 5. Other grades include grade 23, an extra low interstitial version of grade 5 with excellent fracture toughness and grade 29, a ruthenium-enhanced version of grade 5 approved in NACE MR-01-75. This category includes several proprietary grades not included in the ASTM standards, such as T-6-2-2, Ti-6-2-4-6 and Ti-550, that nevertheless are very important alloys with applications in aerospace.

Near beta and beta alloys

These are microstructurally predominantly a beta structure and can be heat treated to high strength in larger sections than the alpha-beta alloys. Their main applications are in aerospace, for example in forgings used for aircraft

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More widely used alloys	Attribute/applications
CP Ti grades 1 and 2	Corrosion resistance with ease of fabrication and welding
CP Ti grades 3 and 4	Corrosion resistant higher strength than grade 2. Grade 4 used for prosthetics
Ti/Pd grades 7,11,16 and 17	Enhanced corrosion resistance
Ti-0.8Ni-0.3Mo grade 12	Higher strength than CP but with enhanced corrosion resistance
Ti-10V-2Fe-3Al (10-2-3)	High-strength heat-treatable beta forging alloy
Ti-15Mo-3Nb-3Al-0.2Si (grade 21)	Oxidation- and corrosion-resistant beta sheet alloy
Ti-15V-3Cr-3Sn-3Al (15-3-3-3)	High-strength heat-treatable beta sheet alloy
Ti-3Al-8V-6Cr-4Zr-4Mo (Beta C) ASMT grade 19	Beta alloy with well-established aero and industrial applications
Ti-4.5Fe-6.8Mo-1.5Al (LCB)	Lower-cost beta alloy for engineering use including springs
Ti-4Al-4Mo-4Sn-0.5Si (551)	High-strength airframe alloy
Ti-5.8Al-4Sn-3.5Zr-0.7Nb (834)	Advanced aero engine alloy, creep and oxidation resistant
Ti-5Al-2.5Sn ELI (and grade 6)	Good combination of properties for cryogenic application
Ti-6Al-2Sn-4Zr-2Mo (6-2-4-6)	Creep- and oxidation-resistant aero engine alloy
Ti-6Al-7Nb (367)	Medical and prosthetic applications
Ti-6Al-4V ASTM grade 5, including Ti-6Al-4V ELI (grade 23) and grade 29	Workhorse general purpose high-strength alloy
Ti-Al-2.5V grade 9 (and with Pd Gr.18)	'Half 6–4' more easily fabricated, medium-strength alloy

Table 2 Summary of some available titanium alloys and their applications

undercarriage, and include grade 19 (Ti Beta C) and Ti-10-2-3.

Table 2 lists a summary of some available titanium alloys and their applications.

Mechanical properties of titanium alloys

Table 3 lists the mechanical properties of some titanium alloys, compared with other alloy systems. It is clear that titanium alloys have a strength-to-weight ratio that exceeds that of high-strength alloys such as BS S155 (300M) NiCrMoV steel and high-strength lightweight alloys such as 7075 T6 aluminium.

Titanium grade 5 is the most widely used high-strength titanium alloy, and its mechanical properties, both static and dynamic, are affected by heat treatment. **Table 4** lists its static mechanical properties in more detail. This alloy is hardenable in sections up to 4 in, but moderate hardenability means that thicker sections have reduced strength.

Fatigue and crack propagation in titanium alloys is affected by microstructure, surface condition, microstructural texture and the environment, so it is advisable

Material	Ultimate tensile strength at 20°C: MPa	Yield strength at 20°C: MPa	Density: kg/l	Yield strength/weight ratio at 20°C: MPa/kg/l
Titanium grade 2, 7,16, 26	345	275	4.51	61
Titanium grade 23, 29	860	759	4.42	172
Titanium grade 5, 24	900	828	4.42	187
Ti-6-2-4-6	1170	1100	4.65	236
Ti-550	1100	960	4.54	211
Ti-10-2-3	1200	1100	4.65	236
Stainless steel 13% Cr AISI 410	510	350	7.72	45
Stainless steel 316L	485	173	7.94	22
817M40 (AISI 4340) alloy steel	1200	940	7.8	120
BS S155 (300M) NiCrMoV steel	1900	1550	7.8	198
Monel 400	480	200	8.83	23
Aluminium 7075 T6	530	505	2.81	180
Inconel 625	830	415	8.44	49

Table 3 Typical mechanical properties for titanium and other metallic alloys

Product: in	Condition	Specification	Dir.	Ultimate tensile strength: MPa	Yield strength: MPa	Elongation: %
0.025–1.000 sheet and plate	Annealed	ASTM B265	L & LT	895	820	10
⩽3.00	Annealed	ASTM B348	L & LT	895	820	10
⩽4.00	Annealed	Mil-T-9047G	All	895	820	10
>4.00–6.00	Annealed	Mil-T-9047G	All	895	820	10
<0.500	STD	Mil-T-9047G		1130	1060	10
>0.500–1.000	STD	Mil-T-9047G		1100	1030	10
>1.000–1.500	STD	Mil-T-9047G		1060	1000	10
>1.500–2.000	Rd, sqr., hex.	Mil-T-9047G		1030	965	10
>2.000–3.000	Rd, sqr., hex.	Mil-T-9047G		965	890	10

Table 4 Mechanical properties of grade 5 at 20°C

that the engineer produce data specific to the material and the environment.

Corrosion resistance

Titanium's corrosion resistance results from the presence of a thin yet tenacious oxide layer on its surface. This film forms spontaneously and is always present in oxidising environments and is stable and self-healing if damaged. Titanium therefore offers corrosion resistance that varies with the environment. The types of environments that titanium is usually exposed to may be oxidising, reducing or marine.

Oxidising environments

For titanium, 'oxidising' means conditions where a critical minimum level of oxidising agent is present in the media, under conditions where it can react with titanium preferentially to repair or inhibit further breakdown of the oxide film. This covers a very wide range of chemicals, mixtures, process conditions and products. Oxidising chemicals to which titanium is resistant include, nitric, chromic, perchloric and hypochlorous acids.

In practice, air, moisture, moist gases, water, even heavily polluted brackish water and seawater, will support the oxide film. Deaeration of an aqueous media does not affect the corrosion resistance of titanium, because water alone supports the oxide film. Only traces (e.g. ppm) of moisture or oxygen are required to maintain passive behaviour in most environments. Notable exceptions are chlorine gas, red fuming nitric acid and methanol, where specific concentrations of moisture from 1.5% up to 10% are required to support the oxide film on titanium alloys under the worst conditions of environment and temperature.

Reducing environments

Pure reducing acids such as hydrochloric, sulphuric and phosphoric acids will attack titanium in proportion to concentration and temperature. Titanium is inhibited from attack, and its oxide film supported, by the presence of limited concentrations of oxidising ions such as Fe^{3+}, Cu^{2+} and others shown in **Table 5**. Inhibition becomes less effective as acid concentration and temperature increase. No inhibition is available to prevent breakdown of the titanium oxide film by hydrofluoric acid and acidic fluorides.

The corrosion resistance of titanium and its alloys in crevices or under deposits (in strongly reducing conditions and in the absence of oxygen or oxidisers) can be enhanced by alloying with palladium, ASTM grades 7, 11, 16, 17, 18, 20, 24, or ruthenium, ASTM grades 26, 27, 28, 29.

Marine environments

Titanium is highly resistant to chloride-containing marine waters, as can be seen in **Table 6**.

Problems may occur if titanium is coupled to a less noble material, such as carbon steel, particularly if the ratio of the surface area of the titanium to the other material is large. This may result in galvanic corrosion, whereby steel bolts embedded in titanium for example may corrode at an accelerated rate. Where the use of dissimilar metals cannot be avoided, it is best to either isolate the titanium using rubber or use a metal of intermediate nobility, such as 304 stainless steel.

Titanium in construction

In architecture and construction, titanium is commonly used as external cladding for buildings. It is in some respects

Moderate inhibition	Strong inhibition	Very strong inhibition
Oxygen, Ni^{2+}, OCl^-, picric acid, nitro- nitroso- and quinone organics	ClO^{3-}, cupric, ferric and mercuric ions, Ti^{4+}, Te^{4+},Te^{6+}, Se^{4+}, Se^{6+}	Bromine, chlorine, ClO^{4-}, Cr^{6+}, Mo^{6+}, Mn^{6+}, V^{5+} ions of Au, Ir, Pd, Pt, Ru

Table 5 Inhibition environments

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Mode of corrosion	Copper-based alloys	Stainless steel 316	Stainless steel 6 Mo and duplex	Titanium alloys
General corrosion	Resistant/susceptible[1]	Resistant	Resistant	Resistant
Crevice corrosion	Susceptible	Susceptible	Susceptible (>25°C)	Resistant (<80°C)[2]
Pitting attack	Susceptible	Susceptible	Resistant	Immune
Stress corrosion	Susceptible[1]	Susceptible (>60°C)	Resistant	Resistant[3]
Corrosion fatigue	Susceptible	Susceptible	Susceptible	Immune
Galvanic attack	Susceptible	Susceptible	Resistant	Immune
Microbiological corrosion (MIC)	Susceptible	Susceptible	Susceptible	Immune
Weld/HAZ corrosion	Susceptible	Susceptible	Susceptible	Resistant
Erosion corrosion	Susceptible	Resistant	Resistant	Highly resistant

1 Dependent on pollution level/seawater chemistry.
2 Grades 7, 11, 12, 16, 17, 20, 21, 24, 28, 29 resistant to at least 200°C.
3 Standard grade 5 has finite susceptibility, grade 23 (ELI) has improved K_{1SCC} values.

Table 6 Corrosion resistance of titanium and other alloys in marine environments

aesthetically similar to 304 or 316 stainless steel, being a silvery-grey colour and is non-magnetic. Commercially pure grade 1 (ASTM B265) is most commonly specified for architectural applications. It is available in sheet, strip and coil, plate, composite steel plate, bar, extrusions and castings. Typical sheet sizes are 1220 mm in width. The physical properties of titanium compared to stainless steel are noted in **Table 7**.

Titanium's density of two-thirds that of steel means that even large areas of roofing can be carried on less massive structural members, with corresponding cost savings. The Guggenheim Museum (**Figure 8**) for example is clad with 32 000 m^2 grade 1 titanium sheet, manufactured to the architect's specification with a deliberately introduced ripple and soft textured finish. Roofing and cladding sheet thicknesses are typically 0.3–0.4 mm, weighing 1.35 to 1.8 kg/m^2. This compares with 4.6 to 7.7 kg/m^2 for 0.6–1.0 mm stainless steel sheet and 4.5 to 9.0 kg/m^2 for 0.5–1.0 mm copper sheet.

The low coefficient of thermal expansion of titanium is virtually equal to that of glass and granite and close to that of concrete. Consequently, the thermal stress on titanium is very small and may allow direct bonding of glass to titanium. Longer lengths of roofing sheet can be laid in titanium than in other metals, thus reducing installation cost.

Titanium, with a melting point of 1660°C, has been certified as a 'non-combustible material' for roofing and cladding by the Japanese Ministry of Construction. Uninsulated titanium can withstand fire tests in which the metal temperature is sustained at 1100°C. As with all metals, however, tensile strength decreases rapidly at elevated temperatures. Titanium is also 100% recyclable. Some investigative work has been carried on titanium as a structural member, but the problems of welding large sections of material economically under an inert gas atmosphere and the high initial raw material cost, have not yet been overcome.

Welding

Titanium can only be welded to titanium, but this can be done using conventional gas shielded techniques such as gas tungsten arc welding (GTAW), also known as tungsten inert gas (TIG) welding. The advantages of GTAW/TIG welding are that is can be a manual or mechanised process and can produce high-quality welds with no weld spatter. Care must be taken to use proper argon gas shielding with a gas shroud to ensure that no oxygen comes into contact with the weld pool, as this can cause brittleness of the weld. Equipment used for stainless steel seam welding can be used to weld titanium and the welding can be performed without an inert shielding gas. Studs for

	Grade 1 Ti	Grade 2 Ti	Grade 5 Ti	304 stainless steel
Density, g/cm^3	4.51	4.51	4.42	7.9
Coefficient of thermal expansion (20–100°C), $m\,m^{-1}\,°C^{-1}$	8.6×10^{-6}	8.6×10^{-6}	9.0×10^{-6}	16.6×10^{-6}
Thermal conductivity, $W\,m^{-1}\,K^{-1}$	21.97	21.79	6.6	16.3
Young's modulus, GPa	110	110	115	200

Table 7 Physical properties of titanium alloys compared to 304 stainless steel

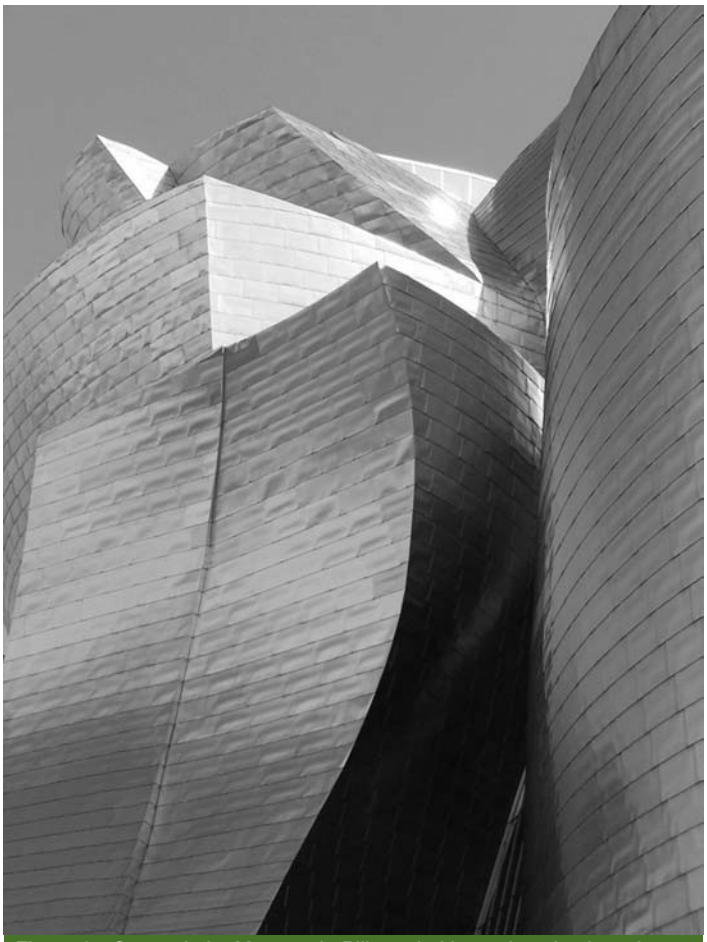

Figure 8 Guggenheim Museum in Bilbao clad is grade 1 titanium sheet (courtesy of TIMET)

mounting can be capacitance discharge welded on to the back of 0.4 or 0.6 mm sheet with little panel distortion and no read-though. Brazing and adhesive bonding can also be used giving acceptable joint strength and durability.

Forming

Titanium grades 1 and 2 can be formed on equipment used for stainless steel. The lower Young's modulus of titanium compared waith stainless steel means that there is a greater degree of spring-back that has to be taken into account. The room temperature ductility of titanium is generally less than that of other common structural metals, which means that titanium may require more generous bend radii and has lower stretch formability.

Aesthetics

In architectural applications (see **Figures 9** and **11**), the aesthetic appearance of titanium is a very important factor and has led to it being used in many high-profile projects such as the Guggenheim Museum in Bilbao (**Figure 8**), the National Scottish Science Centre in Glasgow, the

Figure 9 Crystal® finish on titanium (courtesy of TIMET)

Scheepvaart Museum in Amsterdam and the Vinos Herederos del Marques de Riscal winery in the Rioja region of Spain (**Figure 12**). The oxide layer on the surface of the titanium is colourless at its normal thickness, but thickening by anodising can produce a range of spectacular refractive colours, whose attractiveness can be enhanced by pre- treating the metal surface. Effects from a soft matte finish to bright, near gleaming reflectivity are possible.

The colours are inherent in the material and do not alter or fade unless subjected to physical abuse. In applications where the titanium will be in repeated contact with people's hands for example, a sol-gel coating can be applied which confers improved abrasion resistance and also makes the titanium surface 'fingerprint' proof. These coatings can be sprayed on and heated to a moderate temperature for curing, to produce a hard, ceramic-like transparent surface, so application must be carried out prior to installation. Mill

Figure 10 Private residence, Holland; architect: Alberts & Van Huut Architecten (courtesy of TIMET)

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Figure 11 Private residence, Boston, Massachusetts; architect: Thomas Peter Dolle AIA (courtesy of TIMET)

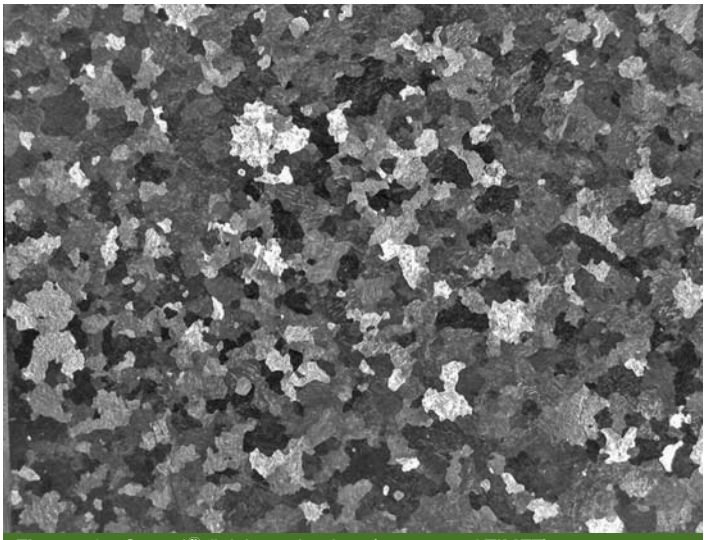

Figure 13 Crystal® finish on titanium (courtesy of TIMET)

finishes and embossed finishes similar to those obtained with stainless steel are readily available, in addition to more customised finishes such as TIMET's Crystal® (**Figures 9** and **13**).

Where incorrect installation takes place, for example by removing the protective PVC (polyvinyl chloride) film before other trades are finished in the area, this may result in scratching or contamination of the surface. A more aggressive cleaning procedure may then be required, in which the oxide layer is chemically removed to restore the clean titanium surface. Proprietary cleaning agents are available but care should be taken in their use as they may contain aggressive chemicals. It is advisable to treat each project on a case-by-case basis in order to first ascertain the cause of any discoloration prior to treatment.

Cost and sustainability

The structural weight reductions that can be achieved when titanium is used, coupled with very low maintenance costs and over 100 years' corrosion-free service, can, with appropriate design, give a lower overall building cost. In competition with stainless steel and copper, titanium has on occasions been the lowest first-cost option. A study by the Japanese Titanium Society compared the lifetime cost of polyvinylidene fluoride PVDF-coated stainless steel to titanium and titanium was seen to be the lower cost option over the longer term, as is illustrated in **Figure 14**.

Titanium ore, as seen in **Table 1**, is commonly available and electricity used in the processing of the ore to produce titanium metal can be sourced from renewable sources. One North American producer, for example, uses electricity from the Hoover dam. Elements left over from the Kroll process are fully recycled. The inherent corrosion resistance of titanium means that components can have very long design lives, measuring in the centuries, under some conditions.

Figure 12 Vinos Herederos del Marques de Riscal winery

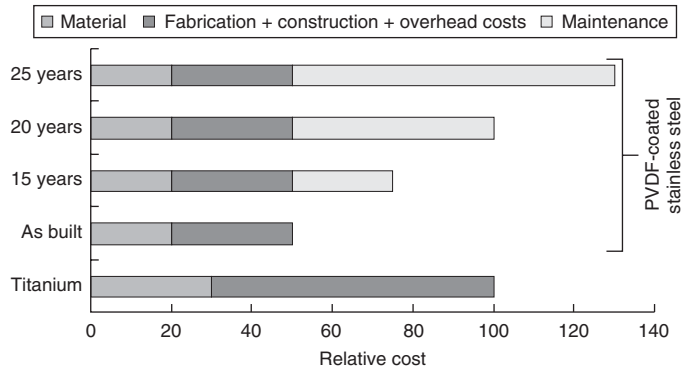

Figure 14 Comparison of lifetime cost of titanium cladding vs PVDF-coated stainless steel

Maintenance

Like all materials exposed to the environment, titanium can accumulate residue from the atmosphere and from rain, thereby altering the refractive properties of the oxide layer and altering the colour of the titanium. In order to maintain the original appearance, it is advisable to plan a cleaning programme with the help of the titanium supplier. This could be in the form of a simple power-wash using standard window-cleaning services, following the customary protocol of wash, rinse and squeegee/wipe dry.

Emerging technologies

Work is progressing on bypassing the conventional route of titanium sponge production and the inherent cost associated with the traditional Kroll process. At the last count there were a least 15 such processes in various stages of development and new ones being announced regularly. The FFC Cambridge (UK) and Armstrong (US) processes are the closest to commercialisation.

If successful, these offer the potential to produce titanium metal powder that can be either directly consolidated using techniques such as laser sintering to produce near net shape parts, or can be used as a raw material for melting in a VAR as sponge is currently.

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Chapter 45

Structural steel

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This chapter concentrates on the specification of hot-rolled and tubular steel structures to BS EN 10025 and BS EN 10210 and their corresponding design standards, BS EN 1993: Eurocode 3 and formerly BS 5950. It also covers other types of steel, such as cold-formed steel to BS EN 10346 and stainless steel to BS EN 10880. The manufacture of steel in terms of metallurgy and process is reviewed which links to the ways of satisfying toughness and weldability criteria. A description of the specification of different modern steel grades is presented and is linked to former standards. Different types of steel sections are presented and their manufacturing process is described. A detailed section on stainless steel reviews the modern steel grades, and particular issues as regards mechanical properties, environmental conditions, connections, etc. Other sections in this chapter provide information on corrosion protection, fire protection and site installation, which are important aspects of steel construction.

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Introduction

A wide range of steel components is available to the architect and designer from:

- hot-rolled sections such, as I, H, C and L shapes
- tubular sections of circular, square, rectangular and now oval shapes
- fabricated beams made by welding plates
- light steel components made from strip steel
- stainless steel components
- modular units, made primarily from light steel components.

This chapter concentrates on the specification of hot-rolled and tubular steel structures, but the principles are applicable to a 'kit of parts' of steel components (Trebilcock, 2004). Indeed, in many buildings, I sections are used for beams, H sections for columns, tubular sections for bracing, fabricated sections for primary beams or transfer beams supporting columns, light steel sections for infill walls, and modular elements for plant rooms and toilets. Site connections are usually made by bolting, although welded connections may be preferred for factory-made connections.

Hot-rolled steel sections

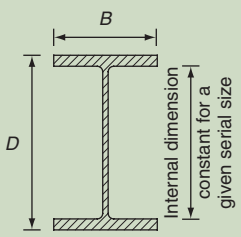
A wide range of standard steel sections is produced by hot rolling, from which designers can select the profile, size and weight appropriate to the particular application. **Table 1** illustrates the range of 'open' sections used in the UK, which are universal beams (UB), universal columns (UC), parallel flange channels (PFC) and angle sections. Corus *Advance* sections are now designated as UKB, UKC, UKPFC, etc. for applications in the UK and new sections have been introduced by Corus to extend the standard range.

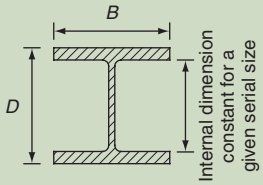
In continental Europe, IPE and HE sections are generally used. IPE signifies I beam sections, which are similar to UB sections, and HE signifies H sections, which are similar to UC sections. However, there are fewer weights of these continental sections in a given section depth than for UKB/UKC sections. In the USA, W and WF sections are used. W sections are similar to UKB sections and WF (wide flange) are similar to UKC sections.

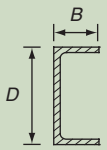
Modern steel sections have parallel flanges. For example, in Europe, parallel flange channels (PFC) have largely replaced channels with tapered flanges. The 'serial size' refers to the designated section depth and width in which there are a range of weights of section. However, it should be noted that the UKB or UKC section designation, such as $406 \times 140 \times 39$ kg/m, refers to the *approximate* depth, width and weight, and exact dimensions should be obtained from standard tables.

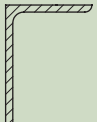
The serial size of UKB and UKC sections varies in increments of approximately 50 mm depth for the shallower sections, and 75 mm for the deeper sections. Within each serial size, the designer may choose from a number of different sections of similar height. The standardisation of steel sections has also led to the adoption of standard connections, which have become familiar within the industry (British Constructional Steelwork Association).

The advantages of these 'open' sections are:

Universal beam (UKB)	Nominal dimensions: mm	
	D	B
	203	102 and 133
	254	102 and 146
	305	102, 127 and 165
	356	127 and 171
	406	140 and 178
	457	152 and 191
	533	165, 210 and 312
	610	178, 229 and 305
	Deeper and shallower UKB sections are available but are not listed here	

Universal column (UKC)	Nominal dimensions: mm	
	D	B
	152	152
	203	203
	254	254
	305	305
	356	368 and 406

Parallel flange channel (UKPFC)	Nominal dimensions: mm	
	D	B
	100	50
	125	65
	150	75 and 90
	180	75 and 90
	200	75 and 90
	230	75 and 90
	260	75 and 90
	300	90 and 100
	380	100
	430	100

Equal angle	External dimensions of equal angle
	25, 30, 40, 45, 50 60, 70, 80, 90, 100 120, 150, 200, 250

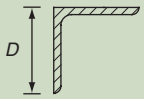
Unequal angle	External dimensions of unequal angle
	Various sizes from: 40 × 25 to 200 × 150 Including common sizes of: 50, 100 × 75, 150 × 75, 200 × 100

Table 1 Advance hot-rolled sections (UKB, UKC, UKPFC and L)

- Wide range of section sizes and weights leads to economic design as beams and columns.
- Fabricators are familiar with their use and these sections are easily available.
- Standard connection details have been prepared and connections or attachments can be made easily.
- Fabrication costs are minimised.
- A range of section weights is available in one serial size without affecting the beam depth or column size significantly in taller buildings.
- Openings through the sections, or notched beams can be made easily.
- Member capacities are given in standard tables.

Table 2 shows the breakdown of costs in a typical structural framework of a building. It is apparent that only 30% of the cost associated with a steel frame relates to the material itself. Costs can increase significantly if excessive fabrication or detailing of stiffeners and connections create a demand for increased labour time. For example, using a heavier steel section is generally cheaper than using a lighter section that has to be stiffened at its connections.

Tubular steelwork

The use of tubular steelwork creates a wide range of architectural opportunities in internal or external applications. The word 'tubular' has come to mean applications using all forms of structural hollow sections, rather than just circular sections. Tubular sections are available as circular hollow sections (CHS), and square or rectangular hollow sections (SHS and RHS respectively), and more recently oval sections. SHS can also be used as the generic title 'structural hollow sections' and, in the UK, Corus' hot-finished SHS have the trade name *Celsius*. More detail on the methods of manufacture are presented later.

Table 3 defines the common SHS section sizes – all tubular sections have *exact* external dimensions for detailing purposes. Oval sections are produced with a 2:1 aspect ratio in sizes from 150 to 500 mm in their longer dimension.

The economic factors that influence the use of tubular construction are their:

- aesthetic appeal, which is often due to their apparent lightness of the members, particularly for circular and oval sections
- torsional resistance (hollow sections are good at resisting torsional effects due to eccentric loading)

Item	Man-hours per tonne	Total % of cost
Materials	3–4	30
Fabrication	8–12	45
Erection	2–4	15
Protective treatment	1–2	10
Total	14–22	100

Table 2 Typical proportion of cost and man-hours per tonne in steel fabrication in buildings

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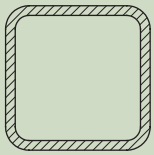
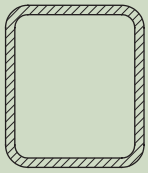
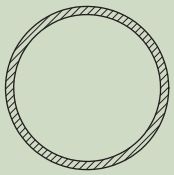
Square hollow section	External dimension of square section: mm
	40, 50, 60, 70, 80, 90 100, 120, 140, 150, 160 200, 250, 300, 350, 400
Rectangular hollow section	**External dimension of rectangular section**
	Depth (mm) × width (mm). Various sizes from: 50 × 25 to 500 × 300 Including common sizes of: 100 × 50, 150 × 100, 200 × 100, 250 × 100, 250 × 150, 300 × 200, 400 × 200, 450 × 250
Circular hollow section	**Size of circular hollow sections**
	Various sizes from: 21.3 mm diameter to 508 mm diameter Including common sizes of: 114.3, 168.3, 193.7, 219.1, 244.5, 273, 323.9, 355.6, 406.4, 457
Oval hollow section	**Sizes of oval sections**
	150 × 75, 200 × 100, 250 × 125, 300 × 150, 400 × 200 and 500 × 250

Note: External dimensions are constant for a given serial size in all hollow sections.

Table 3 Sizes of structural hollow sections

- compression resistance for use as columns or bracing members (tubular sections are very efficient in compression due to their reduced slenderness)
- bending resistance of slender sections (if a beam is unrestrained throughout its length)
- efficiency under combined bending and torsion, such as in structures curved on plan
- reduced fire and corrosion protection costs (because of the lower surface area of tubular sections).

When using larger CHS, for example in long-span trusses, it is important to identify fabricators with specialist profiling equipment who can make the connections between the chords and web members efficiently. This is particularly important for more complex assemblies, such as lattice girders of triangular cross-section.

The excellent torsional resistance and stiffness of tubular sections makes them suitable for curved beams and canopies where members curve on plan and possibly also on elevation. Architects have utilised this property by creating tubular spine beams and inclined arches which resist eccentric loading in bending and torsion.

Fabricated sections

Fabricated steel sections are produced by welding steel plates in a factory process. These sections are fabricated to the required geometry and are not standard sections. They are usually economic where:

- The section size can be 'tailor-made' to the particular application and member depth required, for example where long-span primary beams would not be achievable using hot rolled sections.
- Heavy 'transfer' or podium structures are required to support columns or other heavy loads.
- Asymmetric sections are more efficient than standard sections.
- Tapered sections are specified, for example in grandstand canopies.
- Curved members are created by cutting the web and bending the flange into a curve of the required radius.

The use of fabricated sections is mainly as heavily loaded primary beams in a floor grillage. The primary practical consideration is the availability of standard plate sizes and the relative thicknesses of the plates used in the flanges and web of the section. Examples of the use of fabricated members are also illustrated in **Figure 1**.

Cold-formed sections

A variety of cold-formed sections (CFS) are produced and these sections are widely used as secondary members, such as purlins, or in light steel framing for primary structural applications. Typical C sections are illustrated in **Table 4**. Cold-formed sections are produced by cold rolling from galvanised strip steel in thicknesses of 1.2 to 3.2 mm for structural applications. SCI publications 130 and 301 describe the use of cold-formed steel sections and light steel framing in building.

Standard C and Z sections	Depth of C or Z sections (typical): mm
	75, 100, 125, 140, 170 200, 240, 300 Width of 50 to 75mm
Modified C and Z sections	**Steel thickness (typical): mm**
	1.2, 1.6, 1.8, 2.0, 2.4, 3.2

Table 4 Cold formed sections (produced by various manufacturers)

Cold-formed steel sections are used as:

- purlins in roofs and side rails of industrial buildings
- infill walls in primary steel-framed buildings
- floor joists and secondary members in frames
- light steel framing, erected as storey-high wall panels, in housing and residential buildings
- modular units for use in cellular building forms, such as hotels and student residences
- cladding support members and over-cladding or over-roofing systems in building renovation
- roof trusses, particularly prefabricated roof systems.

Steel decking is produced in steel thicknesses from 0.9 to 1.25 mm and is available in two generic forms:

- deck profiles of 45 to 80 mm depth for use in composite construction
- deep deck profiles of up to 225 mm depth for use in slim floor or *Slimdek* construction.

These applications are used in composite construction in which the decking acts composite with the concrete placed on it.

Modular construction

Modular construction uses prefabricated 'volumetric' components, which are generally made from light steel frames, although they often incorporate SHS columns for the corner 'posts'. Modular construction is most economic where the modules can be manufactured repetitively to achieve 'economy of scale', and where the dimensions of the modules are suitable for transportation and installation (3.0 to 4.2 m are typical module widths).

The Peabody Trust's Murray Grove project in London achieved architectural acclaim by being the first major use of modular construction in the social housing sector. The modular nature of the building was softened by using prefabricated balconies, access walkways, and a 'core' lift and stairs structure at the axis of the two wings of the building.

More guidance on the use of modular construction can be found in recent SCI publications P302 and P348. Modular units can be used in more regular framed structures in the form of prefabricated plant rooms, 'clean rooms', bathrooms and toilets, which are often lifted or slid into place on the floor.

Technical characteristics of steel

Steel is a quality assured and dimensionally accurate product which is available in a wide range of components. The following sections present some of the key technical characteristics and design standards for steel as a material, and in its primary components. The use of stainless steel and cast steel may also be considered in architecturally expressive structures.

Introduction to structural steels

Weldable structural steels are defined by British Standard BS EN 10025, which is the UK issue of a European (EN) standard for steel as a structural material. This standard has replaced BS 4360 since 1993. Designations S275 and S355JR to BS EN 10025 have replaced grades 43A and 50B respectively to BS 4360, and refer to the minimum guaranteed yield strength of the material in N/mm^2. Apart from the change in designation, the properties of S275 and Grade 43 steel are essentially the same. Design resistances are based on the yield strength of steel rather than its ultimate tensile strength. The yield strength of steel is approximately 70–80% of its ultimate tensile strength for most grades.

For hot-finished tubular sections, the relevant standard is BS EN 10210 and the most common grade is S355J2H (which replaces the previous grade 50D).

For cold rolled tubular sections, the standard is BS EN 10219 which designates steel in the same way as for hot rolled sections. Cold rolled sections should not be used as a direct substitute for hot rolled hollow sections and it is important that the relevant standard is referred to in the designation, i.e. BS EN 10210 – S355J2H.

For so-called 'weathering steels', the standard is BS EN 10025-5 and the common designation is S355J0W, replacing grade WR50B in the previous standard.

Introduction to design standards

Until withdrawn in March 2010, the design standards used for the design of steel structures in the UK were BS 5950 for buildings and BS 5400 for bridges. BS 5950 had various parts which applied to the use of hot-rolled and cold-formed sections in general building construction, in which the materials are designated as described later.

Eurocode 3 (official designation BS EN 1993, but usually referred to as EC3) is the European standard for steel structures in the EU member states, which are aligned to each country's nationally determined parameters by National Annexes (NAs). Eurocode 3 comprises various parts:

Part 1: General Rules and Rules for Buildings
Part 2: Steel Bridges
Part 3: Masts, Towers and Chimneys
Part 4: Tanks, Silos and Pipelines
Part 5: Piling
Part 6: Cranes

The key parts of Eurocode 3 are Part 1.1 (General Rules and Rules for Buildings) and Part 2 (Steel Bridges) which align to the standards BS 5950-1 and -2, and BS 5400-3

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respectively. The standards refer to the appropriate EN specifications for definition of structural steel grades to BS EN 10025.

There are various subsections of BS EN 1993-1 applying to different types of steel used in buildings and to special issues such as fire resistant design, as follows:

Part 1.1: General Rules and Rules for Buildings
Part 1.2: Structural Fire Design
Part 1.3: Cold Formed Thin Gauge Members and Sheeting
Part 1.4: Stainless Steels
Part 1.5: Plated Structural Elements
Part 1.6: Strength and Stability of Shell Structures
Part 1.7: Strength and Stability of Planar Plated Structures Transversely Loaded
Part 1.8: Design of Joints
Part 1.9: Fatigue Strength of Steel Structures
Part 1.10: Selection of Steel for Fracture Toughness and Through-thickness Properties
Part 1.11: Design of Structures with Tension Components Made of Steel
Part 1.12: Supplementary Rules for High Strength Steel

The approach for the selection of steels for avoidance of brittle fracture is similar in BS 5950-1, BS 5400-3 and Eurocode 3. Fatigue is usually not considered in the design of buildings but can play an important factor in bridge design.

Composite construction, which comprises steel frames acting compositely with concrete or composite floor slabs, was covered by BS 5950-3 and -4 and, now by Eurocode 4 (BS EN 1994-1-1). Eurocode 4 also extends to the use of composite columns and partially encased sections.

Fire-resistant design was covered by BS 5950-8 and now by BS EN 1993-1-2 and BS EN 1994-1-2 for steel and composite construction respectively. However, thermal performance data of fire protection materials and the elevated temperature behaviour of steel or composite structures is obtained from standard BS EN fire tests.

Manufacturing methods for hot-rolled steel sections

Open sections

Hot-rolled steel I, H, C and L sections are produced by continuous rolling of raw steel slabs, blooms and billets in a process in which the steel is sufficiently hot and malleable that it can be formed into shapes dictated by repeated passing through multiple pairs of rolls. For I or H sections, internal rolls define the internal dimensions of the section and thickness of the web, whereas external rolls control the required thickness of the flanges.

The rolling process is described in more detail later. The interrelation between the main steel properties of strength, toughness and weldability is an important aspect of modern steel making.

Although the rolling process is relatively simple in principle, the number of passes through the rolls and the rates of cooling are carefully controlled in order to achieve the correct dimensions and through-thickness properties, particularly for thicker steel flanges.

Manufacturing methods for hollow sections

Tubular steel is the common term given to structural hollow sections (SHS) with either a square, rectangular or circular cross-section (the range of dimensions is given in **Table 2**). These sections are available in a range of sizes and thicknesses. Importantly, the external dimension is constant for the range of thicknesses in a given section size.

Section properties of SHS are given in Corus and SCI publications. *Celsius* is a trade name of Corus used to denote hot-finished tubular sections produced by the electrical weld (EW) process. Jumbo sections can be manufactured by the electrical resistance weld (ERW) process.

The following description of how these sections are made will help to understand how they may be used.

Seamless hollow sections

Seamless tubes may be made by piercing the solid ingot or bar, which is then elongated in a rotary forge (called the Pilger process). In the rotary forge method, circular or tapered fluted ingots are reheated and are pierced with a mandrel using a hydraulic press. The tube is then formed by rolling externally in a helical manner, reducing the thickness and outside diameter to that of the finished tube while the internal diameter remains unchanged.

Seamless hollow sections are rarely used in structural applications, but can be produced in circular form in diameters up to 500 mm.

Welded hollow sections

Welded hollow sections can be produced by a number of different methods. The most common is known as the conversion method in which round hollow sections are produced and then converted to the required shape by re-rolling. The welding can be carried out by butt or continuous welding, electric weld, spiral weld, and submerged arc welding.

A method used less commonly in the UK, but more extensively in the United States and Japan, is the direct or brake press method in which the rectangular or square section is formed by bending the parent plate and then welded by ERW.

Butt or continuous weld process

In the butt or continuous weld process, which is now seldom used, hot-rolled strip is heated almost to welding

temperature and bent into a horseshoe shape forming a nearly closed tube. The strip edges are then heated locally and pressed together to make the weld. The hot tube then goes through sizing rolls which reduce the outside diameter to within the specified tolerance. This method is now only used for sections of up to 48 mm diameter.

Electric weld (EW) process

The majority of hollow sections used in the UK is produced by the electric weld (EW) method, which can produce circular sections from 48 mm diameter up to 508 mm diameter, square sections from 40 mm up to 400 mm and rectangular sections from 50×30 mm up to 500×300 mm. The thicknesses produced depend on the size of the section, and range from 2.5 mm to 6.3 mm in the small sizes up to 6.3 mm to 16.0 mm in the larger sizes.

In the EW process, the strip is progressively formed into a round, nearly closed tube shape and then passes through a high-frequency induction coil which raises the strip edges to fusion temperature. The edges are then pressed together forming a weld without the use of any filler (electrode) material. The round hollow is formed to a diameter which will create the finished section size.

After welding, the external weld flash or bead is removed, but the internal bead is normally left untrimmed. The weld bead is regularly checked for uniformity and integrity.

All shapes of hollow sections can be produced as hot-finished or cold-formed sections. Both processes use hot-finished strip as their feed-stock, and are initially formed into cold round sections and then welded as described above. Cold-formed sections are then finished in their 'cold' or normal state into the desired shape. Hot finished sections are heated and formed into circular, square or rectangular shape while in the normalising temperature range. Hot-finished and cold-formed products are different in their mechanical performance and each has its own design code and product standard.

Forming the finished shape

A circular tube is the starting point for making a circular, square or rectangular hollow section. The circular tube is first heated and then passed through a series of rolls to produce the correct dimensions.

Rectangular and square hollow sections are formed by passing the circular sections through a series of rolls which change the profile shape gradually into the required shape, as indicated in **Figure 1**.

Stretch reduction

Standard 170 mm nominal diameter tubes produced by the EW process can be stretch-reduced to produce other structural hollow section sizes up to the following nominal sizes:

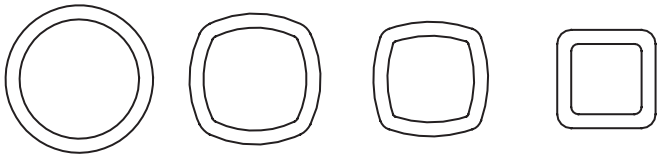

Figure 1 Forming the finished shape of a square hollow section

- circular hollow sections; 140 mm nominal outside diameter
- square hollow sections; 100 mm square
- rectangular hollow sections; 120×80 mm.

In all cases, the maximum thickness of the section by this process is 8 mm. Stretch reduction is the main method of producing sections up to these dimensions.

Hot-finished versus cold-formed sections

Cold formed hollow sections differ slightly in shape and form from hot-finished hollow sections. The principal differences are:

- Square or rectangular cold-formed sections have larger and more rounded corner radii which may give them a less crisp appearance than hot-finished sections.
- The seam weld profile is often more pronounced in cold-formed sections.

Hot-finished hollow sections are supplied to BS EN 10210. Cold-formed hollow sections are supplied to BS EN 10 219. The structural properties of cold-formed sections are less than the equivalent hot finished sections of the same nominal dimensions. Therefore, direct substitution of cold-formed sections for hot-finished sections should not be carried out without careful checking. It is also important that the correct specification is made at the design stage.

Submerged arc welded process

This process is generally used for tubes over 500 mm diameter and up to 2134 mm diameter. Larger sizes are formed from two semicircular rolled plates, smaller sizes from single circular rolled plate with the final weld being made by the submerged arc welding (SAW) process.

Spiral welded process

Spiral welded tubes are made by helically forming of strip which is then welded by submerged arc welding or other CO_2 processes. The method is generally used to produce large-diameter thin-wall tubes, for example, that are used in bored foundation piles.

Oval or elliptically shaped sections

Oval sections are made by adapting the process for circular sections and are produced in a 2:1 aspect ratio for a range of sizes from 150 mm to 500 mm in the longer dimension.

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'Cigar'-shaped columns

These are made out of steel plate/cones which are welded down the seam and then in turn butt welded onto the next piece. These are non-standard items and, as such, are relatively costly, as they have to be made to order for a particular project.

Stainless steel

Grades of stainless steel

Stainless steel is the generic name given to corrosion-resistant steels with a minimum of 10.5% chromium, which oxidises to produce a surface patina, which inhibits rusting. The stability of this thin passive layer increases as the chromium content increases and is further enhanced by alloy additions of nickel and molybdenum.

The grades of stainless steel can be grouped into five basic groups:

- Austenitic stainless steel: these most widely used types of stainless steels are based on 17–18% chromium and 8–11% nickel alloys. They have high ductility and are readily weldable.
- Ferritic stainless steel: they contain 10.5–18% chromium, but less nickel than austenitic grades, and are less corrosion resistant.
- Duplex stainless steels: these steels are generally used where high stresses are to be resisted under severe corrosion conditions, and are often used for bars and pins. They typically contain 21–26% chromium, 4–8% nickel and 0.1–4.5% molybdenum.
- Martensitic stainless steels: these steels have similar structure to ferritic stainless steels and are generally used for less corrosion resistant applications.
- Precipitation hardened stainless steels: these steels are strengthened by heat treatment to very high temperatures and are used for heavy-duty connections, such as bolts.

The most widely used grades are standard austenitic grades 1.4301 (304) and 1.4401 (316), which are specified in BS EN 10088. The designation system refers to: 1 (steel), 43 or 44 (group of stainless steels), 01, etc. (individual grade identification).

The terms in brackets refer to the equivalent ASTM (American) standard, where A304 and A316 are the common grades. The yield strength is typically 200–220 N/mm^2 (which is lower than for carbon steels), although the ultimate tensile strength is 500–700 N/mm^2 (higher than for carbon steels). Duplex steels have 400–460 N/mm^2 yield strength. More details on the types and characteristics of stainless steel are given later.

Stainless steel components

The wide range of stainless steel components are presented in the SCI publication *Architect's Guide to Stainless Steel*. Good examples of the use of stainless steel sections and components are:

- support angles for brickwork
- brackets for cladding systems
- glazing bars and arms
- mullions to support cladding
- stair stringers and steps
- threaded bars for tie rods
- pins for tie rods
- louvres
- various types of finish are possible, including colouring, etching, embossing and shot blasting.

Weathering steels

Weathering steels are a type of steel alloy in which, as weathering occurs, a hard patina is formed on the surface to prevent further corrosion. This patina can vary in colour, depending on the environment, from an orangey brown to dark brown. A recent example of the use of weathering steel is in the cladding and structure of the Baltic Square Tower, Helsinki.

Weathering steels are not suitable for industrial or coastal environments where the patina will not form properly. In the initial stages of weathering, a 'rusty' run-off may occur and adjacent areas of concrete or cladding should be protected. For example, where weathering steels are used for bridge girders, adequate drainage should be provided to prevent staining of concrete abutments. Where weathering steels are used for columns, gravel-filled pockets could be provided at the base to prevent staining of concrete paving.

Use of cast steel

Steel castings have been used for many years, in particular where members meet at a 'node' point, and they offer considerable opportunities for architectural expression. There have been significant improvements in the use of steel castings over the past two decades.

The avoidance of heavily-stressed welds and difficult or expensive fabrication are advantages of castings over welded fabrication. However, the relatively long lead-in time for drawings, pattern making, casting and machining means that the use of castings should be considered at an early stage in the design process.

Castings can also incorporate fixings such as tapped holes for threaded connections, cast-in lugs and cleats to facilitate fabrication or erection. The development of castings in the building industry was hindered for some time by a number of misconceptions with respect to both

materials and casting procedure. Castings were thought to be brittle and unweldable. This is not so, as cast steel can have similar properties to high-grade steel.

Casting design demands input from an experienced foundry, as success depends on choosing a pattern where molten metal will flow at a sufficient rate to fill the mould uniformly, and suitable non-destructive testing is needed to check that the castings do not have large inclusions or surface flaws. This is obviously most important when the castings are highly stressed.

Examples of cast nodes and arms

Castings using steel and other alloys may be used in a wide variety of applications in steel construction, many of which have opportunities for architectural expression, as follows:

- The extended arms or 'gerberettes' of the Centre Pompidou provide the connections to the columns and tension ties. The shape of the 'gerberettes' is designed to resist combined shear and bearing forces and bending effects.
- Ornamental details in façades, as in the bronze supports.
- At the top or intermediate points of masts where cables connect, as used in Stansted Airport terminal.
- The nodes of tension-tie systems, particularly where a number of ties meet. In this case, the castings are designed to transfer the multiple forces smoothly.
- Architecturally exposed connections between columns and beams.
- Brackets or other attachments to members.

SCI P-172 covers the design and use of castings. The form and profile of the casting can reflect the structural action and forces within the connection, or alternatively it can be designed to suit a given architectural motif or shape. The shapes available for castings are generally only limited by the practicalities of pattern making and casting. Due to the bespoke nature of each type of casting, it is more difficult for the costs to be assessed accurately at the design stage but the maximum economy is likely to be achieved if use of the casting is repeated many times.

Other metals used as castings

Iron may also be used for castings. It has a lower melting point than steel and, when molten, flows into a mould more easily. Ductile iron castings are easier to form and, as a result, are generally cheaper than the equivalent steel castings. One disadvantage is that welding to iron castings should be avoided if the iron has a higher carbon equivalent content than steel. However, bolted or pinned connections are easily made and, from the point of view of tolerance, iron is more easily machined than steel.

Castings may also be formed in stainless steel, as were supplied for the Ludwig Erhard Haus, the Berlin Stock Exchange. Bronze castings have also been used.

Structural steel

This section reviews the specification of structural steel and its properties.

Steel specification standard

General

EN 10025: 2004 is the six-part European standard for hot-rolled structural steel. This standard designates grades differently from the previous standards for these steels.

The standard is published in the six parts shown below. It combines what were formerly separate standards into one new standard for the majority of hot-rolled structural steel products:

Part 1: General Technical Delivery Conditions
Part 2: Technical Delivery Conditions for Non-alloy Structural Steels (supersedes EN 10025: 1993)
Part 3: Technical Delivery Conditions for Normalised/normalised Rolled Weldable Fine Grain Structural Steels (supersedes EN 10113: Parts 1 and 2: 1993)
Part 4: Technical Delivery Conditions for Thermo-mechanical Rolled Weldable Fine Grain Structural Steels (supersedes EN 10113: Parts 1 and 3: 1993)
Part 5: Technical Delivery Conditions for Structural Steels with Improved Atmospheric Corrosion Resistance (also known as weathering steels) (supersedes EN 10155: 1993)
Part 6: Technical Delivery Conditions for Flat Products of High Yield Strength Structural Steels in the Quenched and Tempered Condition (supersedes EN 10137: Parts 1 and 2: 1996)

Design to BS EN 1993-1-1: Eurocode 3

The design of steel structures in buildings and related structures is presented in BS EN 1993-1-1: Eurocode 3, which has replaced the various parts of BS 5950 (since March 2010). The UK National Annex to this design standard refers to the product standards BS EN 10025 for rolled sections and plates and to BS EN 10210 for hot-finished hollow sections, rather than using the tabulated values for steel strengths in BS EN 1993-1-1.

Eurocode 3 is different in many respects from BS 5950 but also has some similarities. It uses different (lower) partial factors for loading, but the partial factors for material strengths are essentially the same. It is likely that BS 5950 may still be used to satisfy the requirements of Approved Document A of the Building Regulations and therefore will have extended use for many types of buildings.

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Steel processing

Overview

The different supply conditions for steel are: as-rolled, normalised, normalised rolled and thermo-mechanically rolled (TMCR). Although not normally used for general structural applications, steel in the upper strength range may sometimes be supplied in the quenched and tempered (Q & T) condition.

The different processes and the benefits of each are briefly explained below based on information supplied by Corus RD & T. The aim of all these processes is to produce good toughness and strength properties through grain refinement and at low carbon equivalent value (CEV) levels to maintain weldability. TMCR or Q & T processes for achieve the required strength level at low CEV.

Steel-making and casting

In the UK, steel supplied to BS EN 10025 will normally be provided via the basic oxygen steel-making (BOS) route in cast sizes of approximately 250–300 t. Steel supplied via the electric arc process requires a high percentage of scrap, and this route is not normally used in the UK for general structural steels.

Where steels of enhanced quality are required, the steel from the BOS process can be further refined under vacuum and/or in a ladle with or without external heating. Modern steels for structural applications are usually supplied in the 'killed' condition, i.e. where the steel is treated with at least 0.1% silicon and/or aluminium to remove oxygen.

Influence of alloys on steel properties

Steel is based on iron/carbon alloys with the carbon level in structural steels being generally in the range 0.08–0.25%. In general, increasing the carbon level increases the strength of structural steels. Conversely, it reduces weldability and may reduce toughness. The second major element in structural steels is manganese, which is a strengthening agent. Small additions of other elements, such as nickel, vanadium, and chromium, will also increase the strength of the steel, but to different degrees.

Toughness is usually improved by minimising alloy or residual element content, such as sulphur. Reducing the percentage of carbon also strongly improves toughness. Refining grain size is a very effective way of improving strength and toughness. Certain alloying elements are added to help restrict grain size during the steel processing and heat treatment. Niobium and aluminium are two important elements, which are used for this purpose.

Normal rolling (plates and sections)

Adequate levels of strength and toughness can be achieved when the characteristics of the particular mill are such that under normal processing conditions the finish rolling temperature (FRT) is low enough (<1000°C) to give a modest refinement of the ferrite grain size. In order to develop yield stresses of 355 N/mm^2 and above, small additions of niobium or vanadium, either singly or occasionally in combination, are usually made to CMn steels to provide grain refinement and precipitation strengthening. Notch tough grades are normally silicon 'killed' and aluminium treated.

Controlled rolling (plates and sections)

Controlled rolling is a generic term for rolling procedures in which the temperature and deformation during rolling are controlled to achieve the required microstructure and mechanical properties.

Temperature-controlled rolling (TCR) is a procedure in which the final deformation is carried out in the temperature range 900–950°C. Thermo-mechanical rolling (TMCR) involves significant deformation at temperatures below 950°C. Both techniques lead to a significant improvement in properties, and TMCR is the more effective. It is possible either to roll directly to lower temperatures using a lower start roll temperature or to introduce a 'hold' into the schedule at a high temperature and allow the plate to reach a lower temperature before recommencing rolling.

The improvements in properties are due to a refinement of the austenite grain size during rolling, resulting in the formation of fine ferrite grains from austenite during cooling. To obtain optimum grain refinement requires close control of the reheating temperature prior to rolling as well as the FRT, and total rolling reductions below 950°C are of the order of 50%. Refinement is aided by the addition of micro-alloying elements such as vanadium, titanium and, in particular, niobium.

In general, controlled rolling procedures are only used for sections with flange thicknesses greater than 20 mm and plates of thickness greater than 16 mm. Satisfactory properties for lighter sections and plates are usually achieved by normal processing conditions.

Normalising (plates only)

Normalising will give rise to a substantial improvement in toughness compared with normal rolling. The process involves reheating the plate to 900–950°C followed by natural air or accelerated cooling. The process refines the austenite and ferrite grain size compared to the coarse grained, as-rolled steels. As a general rule, the refinement is almost independent of the original FRT, so that after normalising, the grain size will be similar in different section thicknesses and more uniform across an individual plate.

Normalising rolling (plates only)

Normalising rolling is defined as 'a rolling process in which the final deformation is carried out in a certain temperature range, leading to a material condition equivalent to that

obtained after normalising, so that the specified values of the mechanical properties are retained even after normalising'. Rolling is finished within the temperature window of metallurgical recrystallisation, typically from 950°C down to 900°C.

The mechanical properties are then achieved in the finished rolled condition without need for further heat treatment. The steel compositions employed are similar to those for the normalised product and therefore the performance should be similar. Both the normalising and normalised rolled processes depend upon uniform cooling, which can be readily controlled in plate manufacture.

Quenching and tempering (plates only)

Large increases in strength are obtained by quenching the plate in water or oil from the normalising temperature to form martensite, or bainite, followed by tempering to improve its toughness. Since the rate of heat transfer through steel is limited, the cooling rate of thicker products at the mid-thickness is reduced to the extent that through-hardening may not be possible. Therefore, it is necessary to add alloying elements to increase hardenability. The most effective elements in this respect are boron and molybdenum, the quantity added depending on the product thickness and the strength required.

The toughness of quenched and tempered steels is also very much influenced by grain size and it is therefore important to use a fine-grained steel, the usual grain-refining elements being aluminium and niobium. Copper or nickel can also be added to provide some solid solution hardening although this strengthening mechanism is less important in these steels.

The tempering treatment carried out to soften the martensite can also generate precipitation hardening and additions of approximately 0.03% vanadium can offset the decrease in yield stress that would normally occur due to tempering.

Steel properties

The main properties characterising structural steel are its strength, ductility, toughness and weldability. The steel grades specified in standards reflect an optimisation of the different properties and the cost of production. Where special property combinations are required, they can often be supplied by adjustment to composition or processing. Steel specifications often include the option to request specific properties of the steel in additional to the four main aspects listed above; these include aspects such as cold bending, galvanising and high-temperature properties.

Strength

The strength of steel is determined in a tensile test, where a rectangular or round test piece is loaded in uniaxial tension. The properties defined by this test are:

- Upper yield stress (Re_H) = Load of initial plastic deformation/initial area (N/mm^2)
- Tensile strength (R_m) = Maximum load/initial area (N/mm^2)
- Elongation = (Change in length/initial length) × 100 (%)

An increase in strength is usually associated with a decrease in ductility, as represented by elongation at failure, and an increase in the ratio of yield (Y) stress to tensile (T) strength, the so-called Y/T ratio. Most structural steels are of S275 or S355 grade. S460 steel may be used for bridges or heavily loaded columns, but for beams and members controlled by serviceability conditions, use of higher-grade steel is not normally beneficial.

Toughness

The toughness of individual batches of steel is assessed by the Charpy test, which gives a Charpy impact energy value. This consists of machining a notched bar, cooling it to a specified temperature and recording the energy required to break the bar under impact.

The energy required to break the bar varies with test temperature, from low values at low temperatures to much higher values at higher temperatures. For structural steels, the property change is not linear but the absorbed energy increases abruptly over a very narrow temperature range. Low energy fracture at low temperatures is described as 'brittle' because very little deformation of the sample occurs. In contrast, the high-energy fracture involves substantial deformation and is called 'ductile' fracture. The temperature at which the absorbed energy changes abruptly is called the transition temperature. A low transition temperature is indicative of high toughness since a good level of impact energy will be maintained down to low temperatures.

The most common requirement in BS EN 10025-2 is that steels should achieve Charpy energy absorption of 27 J at a specified temperature between +20°C and −20°C, depending on the steel grade. Steels formerly to BS EN 10113 have guaranteed Charpy impact energies at test temperatures down to −50°C, representing high toughness grades. The value of 27 J originates from the pre-metric requirement of 20 ft-lb, which was originally applied to ship plate. High values of Charpy energy, and hence good performance at low temperature, are achieved by a fine-grained microstructure, low sulphur and generally low carbon levels.

For high-strength steels ($Ys > 355\ N/mm^2$), the achievement of both high strength and toughness requires careful metallurgical design. There are various mechanisms of increasing the steel strength, most of which are related to the addition of different alloying elements, but only grain refinement gives improved toughness at the same time. High-strength steels with improved toughness therefore require either normalising treatment with niobium and/or aluminium additions, or thermo-mechanical treatment

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where fine grain size and strength can be achieved even at low alloy levels.

Processing and composition of the steel are adjusted to improve toughness, and the temperature at which this can be achieved can in some cases be as low as −60°C. For most plates and sections, tests are taken at the rate of one group of three tests per 40 t for each thickness group. Other frequencies of testing, e.g. one test per rolled plate or one test per 5 t, can be specified when required, and indeed, higher minimum impact energy requirements can also be agreed.

Weldability

The required weldability, and hence mechanical properties of modern structural steels, is achieved by obtaining an optimum microstructure through a careful combination of:

- alloy composition
- rolling schedules
- heat treatment.

This and the next section describe the interaction of weldability with other properties, the effect of welding on the properties of steel, possible defects that are encountered in welded joints and advances that have been made to the weldability of steels.

During arc welding of structural steels, additional steel, known as the weld metal, is deposited to form the welded joint. Since welding is carried out at high temperatures, the steel immediately adjacent to the weld deposit experiences a change in microstructure. A consequence of this is that this zone, known as the heat-affected zone (HAZ) may exhibit a different combination of mechanical properties, such as impact toughness and strength.

In the past, it was the general case that weldability decreased with increasing steel strength, but today, high strength can be achieved through lower alloy content and carefully controlled rolling schedules, which offers benefits to impact and fracture toughness. These higher-strength steels also offer the designer greater flexibility when designing welded steel structures. It is normal practice to select a weld consumable which over-matches the parent material in terms of strength.

Even though the microstructure of the parent material is changed during welding in a localised region (the heat-affected zone or HAZ), satisfactory welded joint mechanical properties, equivalent to the steel being joined, can be achieved through the correct selection of the steel, welding process and procedures. Steels specified in accordance with EN 10025, are inherently weldable due to the composition limits. The carbon equivalent value (CEV) is a number giving an indication of a steel's weldability and is a summation of various elements present in the steel, each divided by a number reflecting the influence of that element relative to carbon. Lower CEV implies better weldability, and such steels can be welded cost-effectively, such as with reduced pre-heat, maintaining good HAZ properties.

The above comments apply to all grades in the range S275–S460. While also generally true of higher strength grades above this, there are several additional factors that need to be considered when welding higher-strength steels, which are not covered here.

Additional properties

Steel specifications often include the option to request specific demonstration of other properties of the steel in additional to the four main aspects listed above. Such properties may often be inherent in certain grades but without additional testing, they are unproven. These include aspects such as cold bending, galvanising, through-thickness ductility and high temperature properties.

Due to its inherent ductility, most structural steels can be cold formed. In the case of sections, processes such as cambering and bending in either axis can be used to produce curved sections. Provided this is done correctly, the standard grades of EN 10025 are perfectly suitable for such processing. Recommendations for minimum bend radii in cold bent plates are given in several standards.

Weld designs which produce high levels of through-thickness stress can lead to lamellar tearing, such as in cruciform joints. This phenomenon, usually only found in heavy T-butt and fillet welds, occurs in the parent metal or HAZ parallel to the fusion boundary in steels with poor through-thickness ductility. Good welding practice, coupled with low sulphur steels with proven through-thickness ductility (Z grades), are readily employed to avoid such weld defects, as described later.

Structural steel grades

Structural sections can be produced to the same grades as plates (to BS EN 10025). The majority of sections are produced to grades S275 and S355. Grade S275 is the most commonly ordered grade for smaller sections, while for heavier sections it is grade S355. The production of higher strengths, i.e. grade S420 and S460, is small, and these sections tend to be used in weight-critical structures. Sections are usually supplied in the as-rolled condition, as good levels of mechanical properties can be achieved through this route for most sizes and grades. The use of controlled rolling is less common, but sometimes used for certain strength/toughness combinations.

BS 4360 ceased to be a valid standard in 1993 and Grades 40, 43 and 50A, B, C, D and 50DD were replaced by the grades given in **Table 6**. Although the former Grade 55C was not officially replaced, manufacturers have continued to supply this quality. Weathering steels, such as BS 4360 Grades WR50A, B and C are now covered by

Symbol	Example	Property designated
S	S275	Structural steel
E	E295	Engineering steel
360	E360	Minimum yield strength (R_{eH}) in N/mm^2 at 16 mm
JR	S235JR	Longitudinal Charpy V-notch impacts 27J at +20°C
J0	S275J0	Longitudinal Charpy V-notch impacts 27J at +0°C
J2	S355J2	Longitudinal Charpy V-notch impacts 27J at −20°C
K2	S355K2	Longitudinal Charpy V-notch impacts 40J at −20°C
+AR	S235JR + AR	Supply condition as rolled
+N	S275J0 + N	Supply condition normalised or normalised rolled
Customer options		
C	S235C	Grade suitable for cold forming

Table 5 Symbols used in EN 10025-2: 2004 Non-alloy Structural Steels

BS EN 1035-5, and hot-finished structural hollow sections, excluding weathering grades, are defined in BS EN 10210-1.

The steel designation used in the EN standards consists essentially of four parts plus one or two additions. This is best explained using an example. The former BS 4360 Grade 50D steel is designated as S355J2G3/G4. The 'S' stands for structural steel, '355' indicates the minimum yield stress in N/mm^2 steel of 16 mm thickness, 'J2' implies a longitudinal Charpy energy of 27 J at –20°C and 'G3/G4' specifies the manufacturing process or supply condition as described below.

Steel selection in accordance with design codes

Steel selection is aimed at the avoidance of brittle fracture. Clauses in BS 5950 define a reduced limiting thickness as temperature decreases and severity of loading increases. Alternatively to keep the same limiting thickness for a given loading case, a higher Charpy sub-grade is required.

Limitations are placed on steel thickness since a thicker cross-section of any metal has a reduced tendency to behave in a ductile manner since the local stress state becomes more severe. A Charpy specimen is a standard size but achieving 27 J at a lower temperature means an increase in toughness. This increase in toughness is required to offset the effects of using a thicker cross-section.

Selection of appropriate Charpy grade is made, based on the factors described below.

EN 10025-2: 2004 Grade	Strength at $t = 16$ mm: N/mm^2		Longitudinal Charpy V-notch		BS EN 10025: 1993 Nearest equivalent grade	BS 4360: 1990 Nearest equivalent grade
	Min yield: R_{eH}	Tensile: R_m	Temp: °C	Energy: J		
–[1]	–	–	–	–	S235	40A
S235JR	235	360/510	20	27	S235JRG1/G2	40B
S235J0	235	360/510	0	27	S235J0	40C
S235J2	235	360/510	−20	27	S235J2G3/G4	40D
–[1]	–	–	–	–	S275	43A
S275JR	275	410/560	20	27	S235JR	43B
S275J0	275	410/560	0	27	S235J0	43C
S275J2	275	410/560	−20	27	S235J2G3/G4	43D
–[1]	–	–	–	–	S355	50A
S355JR	355	470/630	20	27	S355JR	50B
S355J0	355	470/630	0	27	S355J0	50C
S355J2	355	470/630	−20	27	S355J2G3/G4	50D
S355K2	355	470/630	−20	40	S355K2G3/G4	50DD
E295	295	470/610	–	–	E295	–
E335	335	570/710	–	–	E355	–
E360	360	670/830	–	–	E360	–

Note:

1 The basic S235, S275 and S355 grades have been removed from the standard because they do not offer a guaranteed minimum impact performance, which is required by the EU Construction Products Directive (CPD 89/106/EC). The lowest grade offered is the JR version for each yield strength variation.

Table 6 EN 10025-2: 2004: Non-alloy Structural Steels: Grades, Mechanical Properties and Nearest Equivalent Grades

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Minimum design temperature

In the UK, the building design temperature is normally −5°C for internal and −15°C for external steelwork. Cold stores, or for applications in other countries, should identify the minimum temperature expected to occur within the design life of the structure. This is typically based on 50-year return period isotherms.

Stress level

This classification is dependent on the level of tensile stresses likely to be experienced by a component as this affects the tendency for fracture. As the level of stress increases, a higher Charpy grade will be needed, if all other factors remaining equal. The stress level which marks the transition which influences brittle fracture is 0.3 × yield strength (0.3 Y_S).

Type of detail

Welded connections in certain tension zones, such as cover plates, can have a higher risk of fracture, thus requiring an enhanced toughness level.

Fabrication methods such as welding, flame cutting, drilled or punched holes can affect aspects such as residual stress and local embrittlement due to heat or cold work. Hence, they affect the required Charpy value.

Plastic strain

Components required to withstand strains beyond yield require enhanced toughness values.

Charpy grade

The Charpy grade used must be such that the thickness of each element, T_1, satisfies:

$$t \leqslant Kt_1$$

where K is a factor which depends on the levels of the parameters listed above and t_1 is the limiting thickness for $K = 1$, which represents the base case of a high stress classification in a welded detail. K is increased for less severe operating conditions, such as lower stress, and is decreased for more severe conditions likely to promote brittle fracture, such as high strain.

The K factor is highly dependent on the amount of fabrication to the element (web or flange) at the point on the member under consideration. In general, in steel-framed buildings, most of the welding, cutting and drilling associated with fabrication will be concentrated at the connections. Considering the connection positions on a member will therefore usually determine the quality of steel required. Values of K for typical cases are given in **Table 7**.

Approach to avoid fracture in Eurocode 3

The general approach in BS EN 1993: Eurocode 3 (EC3) is the same as that of BS 5950 and BS 5400, i.e. the use of a fracture mechanics-based route combined with correlation between fracture toughness and Charpy based on the so-called master curve. The approach considers the same factors as in BS 5950 and BS 5400, but the weighting given to different inputs in EC3 is significantly different, particularly in the area of applied and residual stresses. Also, EC3 covers more issues than in the corresponding clauses of BS 5400 and BS 5950, such as:

- consideration of different potential flaw sizes and geometries
- selection of through-thickness grades (Z grades)
- weld metal and heat affected zone toughness
- use of historical steel production data

Type of detail or location	Components in tension due to factored loads		Components not subject to applied tension
	Stress ⩾ 0.3 Y_{nom}	Stress < 0.3 Y_{nom}	
Modern structural steel	2	3	4
Drilled holes or reamed holes	1.5	2	3
Flame-cut edges	1	1.5	2
Punched holes	1	1.5	2
Welded, generally	1	1.5	2
Welded across ends of cover plates	0.5	0.75	1
Welded connections to unstiffened flanges	0.5	0.75	1

Notes:

1 Where parts are required to withstand significant plastic deformation at the minimum service temperature (such as crash barriers or crane stops), K should be halved.

2 Base-plates attached to columns by nominal welds only, for the purposes of location in use and security in transit, should be classified as plain steel.

3 Welded attachments not exceeding 150 mm in length should not be classified as cover plates.

4 Where abrupt changes in cross-sections coincide with the detail (other than those covered by descriptions above), e.g. service openings, notched cut-outs, etc., the general stress levels should take into account the additional stress concentration effect.

5 The stress considered is the stress excluding residual stresses and stresses from structural integrity checks.

Table 7 *K* Factor for type of detail and stress level to BS 5950 Part 1

Steel designation	Maximum thickness t_1 (mm) when $K = 1$ according to minimum service temperature				
	Normal temperatures		Low operating temperatures		
	Internal −5°C	External −15°C	−25°C	−35°C	−45°C
S275JR	36	20	0	0	0
S275 R	65	54	36	20	0
S275 JL	94	78	65	54	30
S355 JR	25	14	0	0	0
S355 J0	46	38	25	14	0
S355 JL	66	55	46	38	21
S355 K2	79	66	55	46	38

Table 8 Limiting thickness for pates, flats and rolled sections to BS 5950 Part 1

- use of testing to back up selection criteria
- linking of fatigue design approaches with fracture aspects and non-destructive inspection intervals.

Steel selection for avoidance of lamellar tearing

Weld designs that produce high levels of through-thickness stress can lead to lamellar tearing, such as in cruciform joints, This phenomenon, usually only found in T-butt and fillet welds, occurs in the parent metal parallel to the fusion boundary in steels with poor through-thickness ductility.

Lamellar tearing can be avoided by adopting normal good welding practice, but includes the following:

- using a suitable low-sulfur steel
- choosing low-restraint weld configurations
- ensuring the weld fusion boundary is more normal to the short-transverse plane
- minimising the weld size to avoid build-up of residual stresses
- using low hydrogen processes
- careful preheat to reduce hydrogen but not so much that it will increase the amount of contraction across the weld
- buttering with a low strength weld metal, where a design necessitates the use of a weld-type which is known to promote lamellar tearing.

Inspection criteria for non-destructive examination for laminar-type flaws in welded regions are given in BS 5400-6, while fabrication guidance for avoiding the problem is given in Annex F of BS EN 1011-2. The UK National Annex effectively ignores the quantitative method in BS EN 1993-1-10.

It should be noted that lamellar tearing is an historical problem, largely addressed by the development of low-sulfur steels and good welding practice.

Fatigue

It is stated in BS 5950 that fatigue does not need to be considered in buildings unless a structure or element is subjected to numerous stress fluctuations, for example in crane supporting structures. In bridges, however, fatigue can play a major role in the design, and rules are provided in BS 5400-10. A general standard for fatigue design of welded steel structures is BS 7608. Both codes provide S–N design lines (S = stress, N = number of cycles) for fatigue design of different classes of welded joints.

EC3-1-10 includes a quantitative method for determining the required Z grade, which incorporates most of the above factors. Z is calculated from cumulated weighted factors for:

- weld depth
- shape and position of weld
- thickness
- restraint
- preheat.

Weather-resistant steels

Weather-resistant steels comprise a group of high-strength, low-alloy steels containing elements which, under normal atmospheric conditions, have an enhanced resistance to rusting. The first example of a weather-resistant composition was 'Cor-Ten', developed by United States Steel Corporation (now USC Corporation) in the 1930s˘('Cor-Ten' is the trade name of USC). In the 1960s, the potential for using these steels for buildings and bridges was recognised. These steels typically have a total content of 1–2.5% of the alloying elements chromium, copper, silicon and, in some cases, phosphorus. The minimum strengths and Charpy impact requirements for weathering steels are the same as for S275 and S355 grades (weathering steel in S275 grade is not included in BS EN 10025-5).

In the presence of moisture and air, all low-alloy steels have a tendency to rust at a rate which is governed by the access of oxygen to the metal surface. As the process progresses, the rust layer becomes a barrier to the ingress of oxygen and the rate of rust growth therefore slows down. However, the rust layer becomes non-adherent and detaches from the metal surface. The rusting rate therefore progresses approximately linearly over time, the slope being related to the aggressiveness of the environment.

With weather-resistant steels, the process of rusting is initiated in the same way, but the specific alloying elements used produce a rust film which is much less porous and is much more adherent to the base metal. This rust system develops with time, becomes protective by impeding further access of oxygen and moisture to the metal surface and hence reduces considerably the rate of rust growth.

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The significance of the 'P' in S355J0WP grade is that these steels also have an addition of phosphorus. They are difficult to weld and are designed to be produced as strip.

Stainless steel

Stainless steel grades

There are five basic groups of stainless steel, classified according to their metallurgical structure; these are: the austenitic, ferritic, martensitic, duplex and precipitation-hardening groups. The austenitic stainless steels and the duplex stainless steels are generally the more useful groups for structural applications.

Austenitic stainless steels provide a good combination of corrosion resistance, forming and fabrication properties. Duplex stainless steels have high strength and wear resistance with very good resistance to stress corrosion cracking.

The most commonly used grades, typically referred to as the standard austenitic grades, are 1.4301 (widely known as 304) and 1.4401 (widely known as 316). They contain about 17–18% chromium and 8–11% nickel. Grade 1.4301 is suitable for rural, urban and light industrial sites while grade 1.4401 is a more highly alloyed grade and will perform well in marine and industrial sites.

The low carbon versions of these grades are 1.4307 (304L) and 1.4404 (316L). Grades 1.4301 and 1.4401 were formerly made with significantly higher carbon levels which had implications for corrosion behaviour. Modern production of stainless steel is such that these low carbon grades are more commonly produced than the standard grades, and are likely to be delivered, even when steel to the standard grade is specified.

Grade 1.4318 is a low carbon, high nitrogen stainless steel which work hardens very rapidly when cold worked. It has a long track record of satisfactory performance in the railcar industry and is equally suitable for automotive, aircraft and architectural applications. Grade 1.4318 has similar corrosion resistance to 1.4301 or 1.4307 and is most suitable for applications requiring higher strength than 1.4301 where larger production volumes are required. It is procured directly from the mill. It is more expensive than 1.4301, depending on the amount required.

An increasing use of stainless steels for load-bearing applications has led to a demand for 'lean' duplex grades in which the mechanical and corrosion properties of duplex grades are combined with a leanly alloyed chemical composition. In the recent Parts 4 and 5 of EN 10088, the newly developed duplex grade 1.4162 is included. Grade 1.4162 is suitable for many applications in the construction sector with a proof strength in the range 450–530 N/mm^2, a corrosion resistance between the austenitic grades 1.4301 and 1.4404 and a lean chemical composition.

Stainless steel standards

The relevant standard for flat and long products is BS EN 10088, Stainless Steels. It comprises five parts:

- Part 1: Lists of Stainless Steels, gives the chemical compositions and reference data on some physical properties such as modulus of elasticity.
- Part 2: Technical Delivery Conditions for Sheet, Plate and Strip of Corrosion Resisting Steels for General Purposes, gives the technical properties and chemical compositions for the materials used in forming structural sections.
- Part 3: Technical Delivery Conditions for Semi-finished Products, Bars, Rods, Wire, Sections and Bright Products of Corrosion Resisting Steels for General Purposes, gives the technical properties and chemical compositions for the materials used in long products.
- Part 4: Technical Delivery Conditions for Sheet/Plate and Strip of Corrosion-Resisting Steels for Construction Purposes.
- Part 5: Technical Delivery Conditions for Bars, Rods, Wire, Sections and Bright Products of Corrosion-Resisting Steel for Construction Purposes.

The designation system adopted in EN 10088 is the European steel number and a steel name. For example, grade 304L has a steel number 1.4307, where:

1	*43*	*07*
Denotes steel	Denotes one group of stainless steels	Individual grade identification

The steel name system provides some understanding of the steel composition. The name of the steel number 1.4307 is X2CrNi18-9, where:

X	*2*	*CrNi*	*18-9*
Denotes high-alloy steel	100 × % of carbon	Chemical symbols of main alloying elements	% of main alloying elements

Table 9 gives minimum specified mechanical properties of common stainless steels to EN 10088-2. The chemical compositions of these grades are given in **Table 10**. Both austenitic and duplex stainless steels can be assumed to be adequately tough and not susceptible to brittle fracture for service temperatures down to −40°C.

Design values of the mechanical properties are addressed later. More information can be obtained in the Euroinox/SCI publication, *Structural Design of Stainless Steel*.

Stainless steel bolts

Stainless steel bolts are covered by EN ISO 3506, Corrosion-resistant Stainless Steel Fasteners. The specification gives chemical compositions and mechanical properties for

	Grade	Product form[1]	Max thickness: mm	Minimum 0.2% proof strength[2]: N/mm^2	Ultimate tensile strength: N/mm^2	Elongation after fracture: %
Basic chromium-nickel austenitic steels	1.4301	C	8	230	540–750	45[3]
		H	13.5	210	520–720	45[3]
		P	75	210	520–720	45
	1.4307	C	8	220	520 –700	45
		H	13.5	200	520 –700	45
		P	75	200	500–700	45
Molybdenum-chromium-nickel austenitic steels	1.4401	C	8	240	530–680	40
		H	13.5	220	530–680	40
		P	75	220	520–670	45
	1.4404	C	8	240	530–680	40
		H	13.5	220	530–680	40
		P	75	220	520–670	45
Stabilised austenitic steels	1.4541	C	8	220	520–720	40
		H	13.5	200	520–720	40
		P	75	200	500–700	40
	1.4571	C	8	240	540–690	40
		H	13.5	220	540–690	40
		P	75	220	520–670	40
Low-carbon, high-nitrogen austenitic steel	1.4318	C	8	350	650–850	35
		H	13.5	330	650–850	35
		P	75	330	630–830	45
Duplex steels	1.4362	C	8	450	650–850	20
		H	13.5	400	650–850	20
		P	75	400	630–800	25
	1.4462	C	8	500	700–950	20
		H	13.5	460	700–950	25
		P	75	460	640–840	25

Notes: C = cold-rolled strip, H = hot-rolled strip, P = hot-rolled plate.

1 Transverse properties.

2 For stretcher levelled material, the minimum value is 5% lower.

Table 9 Specified mechanical properties of common stainless steels to EN 10088-2

	Grade	Content of alloying element (maximum or range permitted) by weight %				
		C	Cr	Ni	Mo	Others
Austenitic steels	1.4301	0.07	17.5–19.5	8.0–10.5		
	1.4307	0.03	17.5–19.5	8.0–10.5		
	1.4401	0.07	16.5–18.5	10.0–13.0	2.0–2.5	
	1.4404	0.03	16.5–18.5	10.0–13.0	2.0–2.5	
	1.4541	0.08	17.0–19.0	9.0–12.0		Ti: 5xC–0.7[1]
	1.4571	0.08	16.5–18.5	10.5–13.5	2.0–2.5	Ti: 5xC–0.7[1]
	1.4318	0.03	16.5–18.5	6.0–8.0		N: 0.1–0.2
Duplex steels	1.4362	0.03	22.0–24.0	3.5–5.5	0.1–0.6	N: 0.05–0.2
	1.4462	0.03	21.0–23.0	4.5–6.5	2.5–3.5	N: 0.1–0.22

Note:

1 Titanium is added to stabilise carbon and improve corrosion performance in the heat-affected zones of welds. However, except for very heavy section construction, the use of titanium stabilised austenitic steels has been superseded largely by the ready availability of the low carbon grades, 1.4307 and 1.4404.

Table 10 Chemical composition to EN 10088-2

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Grade[1]	Property class	Thread diameter range	Bolts		Nuts
			Ultimate tensile strength[2]: N/mm^2	Stress at 0.2% permanent strain: N/mm^2	Proof load stress: N/mm^2
A1, A2, A3, A4 and A5	50	⩽M39	500	210	500
	70	⩽M24[3]	700	450	700
	80	⩽M24[3]	800	600	800

Notes:

1 In addition to the various steel types covered in EN ISO 3506 under property class 50, 70 and 80, other steel types to EN 10088-3 may also be used.

2 The tensile stress is calculated on the stress area.

3 For fasteners with nominal thread diameters $d > 24$ mm, the mechanical properties should be agreed between user and manufacturer and marked with grade and property class according to this table.

Table 11 Minimum specified mechanical properties of austenitic grade bolts and nuts to EN ISO 3506

fasteners in the austenitic, martensitic and ferritic groups. Alternative materials not specifically covered in the specification are permitted if they meet the physical and mechanical property requirements and have equivalent corrosion resistance.

In EN ISO 3506, bolt and nut materials are classified by a letter: 'A' for austenitic, 'F' for ferritic and 'C' for martensitic. It is recommended that austenitic bolts are used, for which the properties are given in **Table 11** (taken from EN ISO 3506). The letter is followed by a number (1, 2, 3, 4 or 5) which reflects the corrosion resistance; 1 representing the least durable and 5 the most durable.

Steel grade A1 is specially designed for machining. Due to high sulphur content, the steels within this grade have lower resistance to corrosion than corresponding steels with normal sulphur content. Care should be exercised if Grade A1 bolts are being considered.

Steels of grade A2 have equivalent corrosion resistance to grade 1.4301. Steels of grade A3 are stabilised stainless steels with equivalent corrosion resistance to grade 1.4541. (A stabilised steel is one which contains an addition of a strong carbide-forming agent such as titanium, which reacts preferentially with carbon and prevents formation of chromium carbides.)

Steels of grade A4 contain molybdenum and have equivalent corrosion resistance to grade 1.4401. Steels of grade A5 are stabilised molybdenum-bearing stainless steels with properties of grade 1.4571 steel.

Austenitic bolts can be obtained in three ultimate strength levels (known as property classes), see **Table 11**. The condition of the alloy in property class 50 bolts is soft, resulting in the highest corrosion resistance. Property classes 70 and 80 are cold worked and this can affect corrosion resistance slightly. Property class 50 bolts having machined threads may be more prone to thread galling.

Consideration should be given to matching the strength and corrosion resistance of the bolts and parent material. EN 14399 gives rules for the CE marking of bolts.

Mechanical behaviour of stainless steel

The stress–strain behaviour of stainless steels differs from that of carbon steels in a number of respects. The most important difference is in the shape of the stress–strain curve. Whereas carbon steel typically exhibits linear elastic behaviour up to the yield stress and a plateau before strain hardening is encountered, stainless steel has a more rounded response with no well-defined yield stress (see **Figure 2**). Therefore, stainless steel 'yield' strengths are generally quoted in terms of a proof strength defined for a particular offset permanent strain (conventionally the 0.2% strain). **Figure 2** shows typical experimental stress–strain curves.

Stainless steels can absorb considerable impact without fracturing due to their excellent ductility (especially the austenitic grades) and their strain-hardening characteristics.

There are factors that can change the form of the basic stress–strain curve for any given grade of stainless steel, as follows.

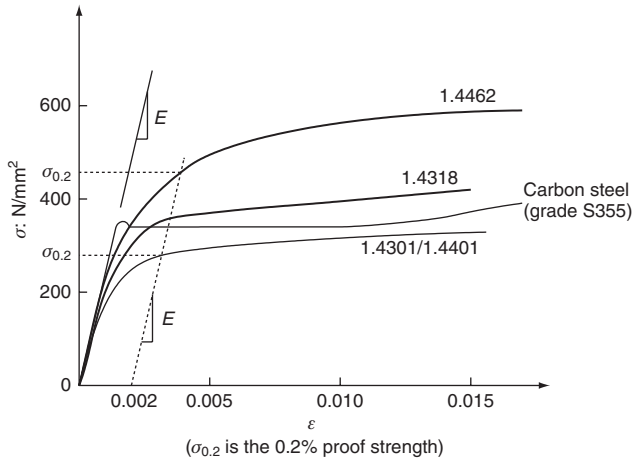

Figure 2 Typical stress–strain curves for stainless steel and carbon steel in the annealed condition (for longitudinal tension)

Cold-worked condition	Minimum 0.2% proof strength[1],[2]	Minimum tensile strength[1],[2]	Available steel grades in the cold-worked condition
CP350	350	700[3]	1.4301, 1.4541, 1.4401, 1.4571
CP500	500	850[3]	1.4301, 1.4541, 1.4401, 1.4571, 1.4318
CP700	700	1000[3]	1.4318, 1.4301
C700	350[3]	700	1.4301, 1.4541, 1.4401, 1.4571
C850	500[3]	850	1.4301, 1.4541, 1.4401, 1.4571, 1.4318
C1000	700[3]	1000	1.4318, 1.4301

Notes:

1 Intermediate 0.2% proof strength or tensile strength values may be agreed.

2 Maximum product thickness for each strength level decreases with strength. Maximum product thickness and remaining elongation are also dependent on the work hardening behaviour of the steel and the cold-working conditions – more exact information may be requested from the manufacturer.

3 Not specified, indicative minimum values.

Table 12 Cold-worked strength levels in BS EN 10088-2 (applicable to material of thickness $\leqslant 6$ mm)

Cold working of stainless steel

Strength levels of austenitic and duplex grades are enhanced by cold working (such as imparted during cold-forming operations including roller levelling/flattening and also during fabrication). Associated with this enhancement is a reduction in ductility but this normally is of little consequence due to the initial high values of ductility, especially for the austenitic stainless steels.

Table 12 gives the cold worked levels specified in BS EN 1993-1-4 which are taken from the European material standard for stainless steel, EN 10088. Cold-worked steels may be specified either in terms of minimum 0.2% proof strength or ultimate tensile strength or hardness. However, only one parameter can be specified.

As stainless steel is cold worked, it tends to exhibit non-symmetry of tensile and compressive behaviour and anisotropy (different stress–strain characteristics parallel and transverse to the rolling directions). The degree of asymmetry and anisotropy depends on the grade, level of cold working and manufacturing route. **Figure 3** shows stress–strain curves for grade 1.4318 cold worked to C850; the compression strength in the longitudinal direction lies below the tensile strength in both the transverse and longitudinal direction (the values given in material standard which is BS EN 10088).

During the cold rolling of a stainless steel section, a proof strength enhancement by a factor of about 50% is typical in the corners of cross-sections. However, the effect is localised and the increase in member resistance is dependent on the type of section. The strength enhancement compensates for any effect due to thinning of the material at cold-worked corners. If advantage is to be made of localised increased strength arising from fabrication, this should be proved by testing.

Subsequent welding of the member will have a partial annealing effect with a consequential reduction in any enhanced strength properties arising from cold working.

Strain-rate sensitivity

Strain-rate sensitivity is more pronounced in stainless steels than in carbon steels; that is, a proportionally greater strength can be realised at fast strain rates for stainless steel than for carbon steel.

Heat treatment

Annealing, or softening, reduces the strength enhancement and the anisotropy.

Other physical properties

A Young's modulus, E of 200 kN/mm^2 is given in BS EN 10088-1 for all standard austenitic and duplex grades used in structural applications. For estimating deflections, the secant modulus is more appropriate and the secant modulus is lower than the initial elastic modulus. For these grades, a value of 0.3 can be taken for Poisson's ratio and 76.9 kN/mm^2 for the shear modulus, G.

The coefficient of linear expansion is approximately 16×10^{-6}/°C for the austenitic grades and differs considerably from that for carbon steel (12×10^{-6}/°C). Where

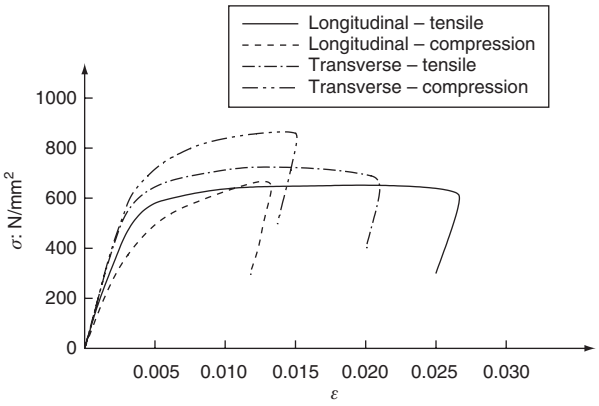

Figure 3 Typical stress–strain curves for grade 1.4318 cold worked to strength level C850

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Steel grade	Location											
	Rural			Urban			Industrial			Marine		
	L	M	H	L	M	H	L	M	H	L	M	H
Basic chromium-nickel austenitic steels (e.g. 1.4301, 1.4307, 1.4541, 1.4318)	✓	✓	✓	✓	✓	(✓)	(✓)	(✓)	×	✓	(✓)	×
Molybdenum-chromium-nickel austenitic steels (e.g. 1.4401, 1.4404, 1.4571) and duplex 1.4362	0	0	0	0	✓	✓	✓	✓	(✓)	✓	✓	(✓)
Duplex steel grade 1.4462	0	0	0	0	0	0	0	0	✓	0	0	✓

Notes:

L Least corrosive conditions within that category, e.g. tempered by low humidity, low temperatures.

M Fairly typical of that category.

H Corrosion likely to be higher than typical for that category, e.g. increased by persistent high humidity, high ambient temperatures, and particularly aggressive air pollutants.

O Potentially over-specified from a corrosion point of view.

✓ Probably the best choice for corrosion resistance and cost.

× Likely to suffer excessive corrosion.

(✓) Worthy of consideration if precautions are taken (i.e. specifying a relatively smooth surface and if regular washing is carried out).

National regulations may contain more onerous requirements.

Table 13 Suggested grades for atmospheric applications

carbon and stainless steel are used together, the effects of differential thermal expansion should be considered in design. Thermal conductivity is typically 15 W/m°C, which is about one-third of that of carbon steels, making it less conductive and better in terms of reducing cold bridging.

Duplex and ferritic grades are magnetic. Where the non-magnetic properties of the austenitic grades are important to the application, care must be exercised in selecting appropriate welding consumables to minimise the ferrite content in the weldment. Heavy cold working, particularly of the lean alloyed austenitic steel, can also increase magnetic permeability; subsequent annealing would restore the non-magnetic properties.

Austenitic grades are also used for cryogenic applications. At the other end of the temperature scale, austenitic and duplex grades retain a higher proportion of their strength above about 550°C than carbon steel. However, the design of structures subject to long-term exposure at cryogenic temperatures or to long-term exposure at high temperatures is a specialist subject. Other stainless steels than those selected here are in most cases better suited for high-temperature applications and further advice should be sought.

Duplex steels should not be used for long periods at temperatures above about 300°C, due to the possibility of embrittlement.

Durability

Stainless steels derive their corrosion resistance from the presence of a passive surface film which, given adequate access to oxygen or suitable oxidising agents, tends to be self-healing when damaged. This oxide film is primarily a consequence of the chromium content of the steel, though the addition of nickel and other alloying elements can substantially enhance the protection offered by the film. In particular, a small percentage of molybdenum is used to improve the pitting resistance of the steel. Corrosion might initiate when the surface oxide film is damaged, possibly by electro-chemical attack or by mechanical damage.

The selection of the correct grade of stainless steel must take into account the environment, the fabrication route, surface finish and the maintenance of the structure. Merely washing down the stainless steel, even naturally by rain, will markedly assist in extending the service life. **Table 13** gives guidance for selecting suitable grades for atmospheric environments.

Caution should be exercised when considering the use of *free-machining* stainless steels for fasteners. The addition of sulphur in the composition of these steels in the austenitic class renders them more liable to corrosion, especially in industrial and marine environments.

Strip steel specification

Standards for strip steel

BS EN 10346: 2004, Continuously Hot-dip Coated Steel Flat Products – Technical Delivery Conditions, presents requirements for continuously hot-dip coated flat products in thicknesses $\leqslant 3.0$ mm made of the steels given in **Table 14**. This EN Standard has recently replaced BS EN 10326 and before that, replaced BS EN 10147. It includes zinc (Z), zinc-iron alloy (ZF), zinc-aluminium (ZA) and aluminium silicon alloy (AS) and applies to strip of all widths and to sheets cut from it ($\geqslant 600$ mm width).

BS EN 10346 is now applicable to:

- continuously hot-dip zinc-coated flat products of low carbon steel strip and sheet for cold forming (formerly to EN 10327)
- electrolytically zinc-coated cold-rolled steel flat products (formerly to EN 10152)

Designation			0.2% proof strength[a]	Tensile strength[b]	Elongation[c]
Steel grade		Symbol for the type of hot-dip coating	$R_{p\,0.2}$: N/mm^2 min	R_m: N/mm^2 min	A_{80}: % min
Steel name	Steel number				
S220GD	1.0241	+Z, +ZF, +ZA, +AZ	220	300	20
S250GD	1.0242	+Z, +ZF, +ZA, +AZ, +AS	250	330	19
S280GD	1.0244	+Z, +ZF, +ZA, +AZ, +AS	280	360	18
S320GD	1.0250	+Z, +ZF, +ZA, +AZ, +AS	320	390	17
S350GD	1.0529	+Z, +ZF, +ZA, +AZ, +AS	350	420	16
S550GD	1.0531	+Z, +ZF, +ZA, +AZ	550	560	–

Notes:
* If the yield point is pronounced, the values apply to the upper yield point (R_{eH}).
* For all grades except S550GD, a range of 140 N/mm^2 can be expected for tensile strength.
* For product thicknesses ⩽0.7 mm (including zinc coating) the minimum elongation values (A_{80}) may be reduced by 2 units.

Table 14 Steel grades and mechanical properties (for thicknesses ⩽3 mm)

- continuously organic coated (coil coated) flat steel products (formerly to EN 10169-1 and ENV 10169-2 and EN 10169-3)
- continuously hot-dip coated strip and sheet of steels with higher yield strength for cold forming (refer to EN 10292).

Classification and designation

The steel grades according to BS EN 10326 are given in **Table 14** and are classified according to their increasing minimum yield strength (R_{eH}). The names of the steel grades are presented in accordance with EN 10027-1.The coating type is defined as follows.

Hot-dip zinc coating (Z)

Application of zinc coating by immersing the prepared products in a molten bath containing a zinc content of at least 99%. In this case, the wide strip of steel is continuously hot-dip coated.

Hot-dip zinc aluminium alloy coating (ZA)

Application of a zinc-aluminium coating by immersing the prepared products in a molten bath which is composed of zinc and approximately 5% aluminium.

Hot-dip aluminium-zinc alloy coating (AZ)

Application of an aluminium-zinc coating by immersing the prepared products in a molten bath which is composed of 55% aluminium, 1.6% silicon and the balance is zinc.

Hot-dip aluminium-silicon alloy coating (AS)

Application of an aluminium-silicon coating by immersing the prepared products in a molten bath, which is composed of aluminium and 8% to 11% silicon.

Coating mass

The total coating mass including both surfaces is expressed in grams per square metre and corresponds to the designations in **Tables 15** and **16**. G275 is the most commonly specified zinc coating thickness for strip steel used in construction, which provides sufficient corrosion protection for applications within buildings. The coating mass is not always equally distributed on both the product surfaces. However, it may be assumed that a coating mass on one face is of at least 40% of the value given in these tables. The period of protection afforded depends on the atmospheric conditions.

Mechanical properties

The strip products are supplied on the basis of the mechanical property requirements in **Table 15**. The tensile test values apply to longitudinal samples and relate to the cross-section without its coating.

Steel grade	Coating designation	Coating finish			
		N	M		
			Surface qualities		
		A	A	B	C
All (+Z)	100	X	X	X	X
	140	X	X	X	X
	200	X	X	X	X
	225	X	X	X	X
	275	X	X	X	X
	(350)	(X)	(X)	–	–
	(450)	(X)	–	–	–
	(600)[a]	(X)	–	–	–

Table 15 Coatings, finishes and surface qualities for zinc coatings (Z)

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Steel grade	Coating designation[a]	Surface qualities		
		A	B	C
Zinc-aluminium coatings (ZA)				
All grades (+ZA)	ZA095	X	X	X
	ZA130	X	X	X
	ZA185	X	X	X
	ZA200	X	X	X
	ZA255	X	X	X
	ZA300	X	–	–
Aluminium-zinc coatings (AZ)				
All grades (+AZ)	AZ100	X	X	X
	AZ150	X	X	X
	AZ185	X	X	X
Aluminium-silicon coatings (AS)				
S250GD + AS S280GD + AS S320GD + AS S350GD + AS	AS060	X	X	X
	AS080	X	X	X
	AS100	X	X	X
	AS120	X	X	(X)
	AS150	X	(X)	(X)

Table 16 Available coatings, finishes and surface qualities for zinc-aluminium coatings (ZA), aluminium-zinc coatings (AZ) and aluminium-silicon coatings (AS)

Coatings

Available zinc (Z) or zinc-iron alloy (ZF) coatings are presented in **Tables 14** and **15**. Thicker zinc coatings limit formability and weldability of the strip steel.

Surface quality (see Tables 15 and 16)

As-coated surface (A)

Imperfections such as small pits, variations in spangle size, dark spots, stripe marks and light passivation stains are permissible.

Improved surface (B)

Surface quality B is obtained by skin passing. With this surface quality, small imperfections such as stretch levelling breaks, skin pass marks, scratches, indentations, spangle structure and zinc runoff marks and light passivation marks are permissible.

Best quality surface (C)

Surface quality C is obtained by skin passing. The better surface should not impair the uniform appearance of a high-class paint finish. The other surface should have at least the characteristics of surface quality B.

The following surface treatment may be applied to the delivered strip steel:

Chemical passivation (C)

Chemical passivation protects the surface against humidity and reduces the risk of formation of 'white rust' during transportation and storage. Local discolouring as a result of this treatment does not impair quality.

Oiling (O)

This treatment also reduces the risk of corrosion of the surface. It is possible to remove the oil layer with a suitable degreasing solvent, which does not adversely affect the zinc surface.

Chemical passivation and oiling (CO)

This combination of surface treatment provides increased protection against the formation of 'white rust'.

Sealed (S)

Application of a transparent organic film coating of masses about 1 g/m^2. This treatment offers additional corrosion protection, especially the protection against fingerprints; it may improve the sliding characteristics during forming operations and can be used as a priming coat for subsequent varnishing.

Phosphated (P)

This treatment improves the adherence and protective effect of a coating applied by the processor. It also reduces the risk of corrosion during transport and storage. Phosphating in conjunction with a suitable lubricating agent may improve workability.

Corrosion protection

The cost of corrosion protection can be as high as 20% of the total cost of fabricated steelwork, and it is therefore important not to over-specify the protective system, while achieving an acceptable life to first maintenance. Corus has prepared various publications; to assist the designer in the protection strategy.

Corrosion of steel can only occur if both oxygen and water are present. The rate of corrosion will depend on the exposure and on the concentration of contaminants (usually chlorides and sulphides) in the atmosphere. Thus permanently embedded steel piles do not corrode, even though they are in contact with water, provided that air is excluded by the impermeability of the soil. Similarly, the interior faces of tubular sections will not corrode, provided they are sealed.

Internal steelwork

Inside heated buildings, only minor and superficial oxidation will occur, except in areas such as roofing or cladding, which may be subject to condensation or water

leakage. Consequently, it is now wide practice to use unprotected steelwork, except where the steel penetrates the building envelope. For largely visual reasons, a thin coat (25 μm) of shop primer may be applied before delivery to site.

Where the steel is exposed to view internally, a decorative coat (often an alkyd) on a suitable primer (usually a zinc phosphate) may be applied. If an intumescent paint is used as fire protection, advice should be sought from the specialist paint manufacturer to ensure compatibility between it and any decorative coating.

Steelwork within perimeter walls is more susceptible to corrosion, particularly if the steel is in contact with or embedded in the outer leaf of a masonry wall. Bitumen, zinc-rich epoxy and pitch epoxy coatings are commonly used in such situations.

For industrial, or other more potentially severe environments, a wide range of options and coatings are available, ranging from use of weathering steels to metallic coatings, such as galvanising or aluminium spray. Provided the individual steel pieces are small, hot-dip galvanising can be a cost-effective option.

Surface preparation

Generally, the better the surface preparation, the better the long-term performance of the coating. Some manufacturers produce coatings which are tolerant of poorly prepared surfaces in order to facilitate maintenance and repainting. In all cases, the method of preparation should be compatible with the coating system and the manufacturers' data sheets should be consulted.

For high-quality coatings, it is essential to remove all the contaminants, rust and mill-scale, which forms when the hot surface of the rolled steel reacts with air to form an oxide.

Surfaces embedded in concrete should be free of mill-scale but may otherwise be untreated, as the alkalinity of the concrete passivates the steel. Treatment on adjacent areas should be returned for at least 25 mm within the concrete. The cover to the embedded steel should be in accordance with the requirements of BS EN 1992-1-1 (which has recently replaced BS 8110).

Various methods are adopted for cleaning steelwork, but the only effective methods for removing all mill-scale and surface containments are blast-cleaning and/or pickling (immersing the steel in acid). Other methods, such as flame cleaning and wire brushing, may be useful for maintenance or preparing steel for use in mild conditions, but they will not remove all the rust and mill-scale.

The method and quality of the surface may be specified according to BS 7079 Part A1. Common specifications are, shot blast to standard SA 2.5, or wire brush to standard St 2. Both grit and shot-blasting are possible, but shot-blasting is preferred for smooth high-quality paint coatings, and grit-blasting is preferred for galvanising and some primers. Where the surface is blast-cleaned, any delay between this cleaning and the application of the first coating should not exceed a few hours in order to prevent further rusting and contamination.

When site painting is adopted, it is important that the steelwork is cleaned before paint application. This may require washing of the steel with a suitable detergent to remove contamination that has occurred during transportation and erection.

For connections exposed to severe environments, it may be necessary to blast-clean the connections before applying the protective system, but this is time-consuming and expensive. Alternatives include the use of galvanised bolts, de-greased after tightening, followed by etch priming and painting to the same specification as the adjacent surfaces.

Type of protection to be used

Paint coatings

There is a large choice of painted coatings suitable for different environments and methods of application. Specific guidance on any particular type of coating is outside the scope of this publication, but detailed guidance can be obtained from manufacturers' catalogues and data sheets. The guidance given by Corus concentrates on a few generic types of system suitable for various applications and environments.

In order to make rational decisions on the type of coating system to be used, it is necessary to understand a few basic concepts about paints and paint systems, as follows.

Paints consist of three basic components:

The pigment – pigments are fine organic or inorganic compounds which provide colour, opacity, film cohesion and sometimes corrosion protection. For example, zinc and micaceous iron oxide (MIO) coatings provide significant levels of corrosion protection.

The binder – binders are usually resins or oils but can be inorganic compounds, such as soluble silicates. The binder is the film-forming component in the paint.

The solvent – solvents are used to dissolve the binder and to facilitate application of the paint. They are usually organic liquids or water. As the paint 'dries', the solvent evaporates into the atmosphere.

Environmental legislation has required paint applicators to control the amount of volatile organic compounds (VOCs) which are released. Paint manufacturers now produce 'high solids' paints (i.e. low solvent) paint and water-based paints which do not release harmful VOCs.

As the solvent evaporates, the thickness of the film reduces. The thickness of the paint coating may therefore be referred to by both its 'wet' and 'dry' film thickness. The wet film thickness can be measured by a comb-gauge

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allowing immediate correction of the film thickness. The dry film thickness can be measured by the use of various types of magnetic or electro-magnetic gauge. Paint thicknesses given in specifications are usually the final required dry film thicknesses.

Paint classification – the most common method of classification of paints is by their pigmentation or their binder type. Zinc phosphate primer, for example, may be used with an alkyd, epoxy or acrylated rubber binder. Care should therefore be exercised when specifying the type of paint, as the performance of zinc phosphate epoxy will be significantly different to that of a zinc phosphate alkyd or zinc phosphate acrylated rubber.

Paint systems – a full painting system will often include a primer, an intermediate coat and a finishing coat. The cost of application of the paint can be a significant factor and the tendency is to use as few coats as possible. In some cases, the same paint can be used for all three coats, and the intermediate coat and finishing coat applied as one high-build (HB) coat. In all cases, the various coats of paint must be compatible with each other.

The following additional points should be considered in the choice of paint system:

The paint system should be compatible with the method of surface preparation and the method of application. For example, site-applied systems would not generally require that the surface is grit-blasted, or that the coating is applied by spray.

The environmental impact of the coating system should be considered. Paints with low levels of volatile organic compounds (VOCs) should be used.

Where paint systems are site-applied, mobile working platforms (MEWPs) or scaffolding may have to be used. Generally, brush application is preferred on site, as sprayed coatings may require air-fed respiratory equipment.

Two-pack epoxies have poor resistance to UV radiation and are highly susceptible to 'chalking'. Over-coating problems can occur when epoxies are applied in two coats, unless they are over-coated before the first coat is fully cured. This is particularly important where a two coat system is partly applied in the shop and partly on site. A 'travel coat' may be required as an intermediate coat between the two epoxy layers. Recoatable epoxies are becoming more common.

Prefabrication primers are usually applied in thin films, in the order of 25 μm (25×10^{-6} m) thick, and their durability is limited. Many modern primers based on synthetic resins are not compatible with manually prepared surfaces as they have a low tolerance to rust and mill-scale. Conversely, many oil- and alkyd-based primers which are tolerant to hand-prepared surfaces cannot be over-coated with some types of finishing coats.

Hot-dip galvanising

In the galvanising process, the steel is first cleaned of rust and mill-scale by blast cleaning and pickling (by dipping in dilute hydrochloric acid containing a rust inhibitor). It is then dipped into a bath of molten zinc at a temperature of about 450°C. At this temperature, the steel reacts with the molten zinc to form a series of zinc/iron alloys on its surface. As the steel element is removed from the bath, a layer of relatively pure zinc is deposited on top of the alloy layers and then solidifies as it cools, often assuming a crystalline metallic lustre, usually referred to as 'spangling'. Hot-dip galvanising is specified to BS EN ISO 1461.

The thickness of the galvanised coating in the hot-dip process is influenced by various factors:

- The size of the element. Thicker, heavier sections tend to lead to thicker coatings;
- The surface roughness. Blast-cleaned surfaces tend to lead to thicker coatings;
- The steel composition. The amount of silicon used in high-strength steel can have a marked effect on the coating weight deposited.

'Hot-dipping' is a bath process and there is obviously a limitation on the size of components which can be galvanised. Double-dipping can often be used when the length of the member exceeds the size of the bath. Some distortion of lighter fabricated members can be caused by differential thermal expansion and contraction, and by relief of residual stresses in the member.

For sections greater than 5 mm thick, a minimum average zinc coating weight of 610 g/m^2 is required, equivalent to a zinc coating of 85 μm thickness. This coating will offer sufficient protection for over 30 years in a clean rural environment, and between 10–25 years in marine and urban situations.

Newly galvanised surfaces are difficult to paint because of adhesion problems, unless an etch primer or mordant wash is used. Due to the smooth surface created by galvanising, a special etching primer or 'T' wash or light 'sweep' blasting will be required before painting, before the final two coats are applied on site.

Connections between galvanised components should generally be bolted rather than welded, due to the poor quality of welded joints and the fumes produced during the welding process.

Galvanising of tubular members

Certain special requirements for galvanising tubular sections are presented in BS EN 10240, and are noted below.

Filling, venting and draining points should be provided in tubular members. Holes not less than 10 mm diameter must be provided in sealed tubular sections to allow access for

Size of tubular section: mm	Minimum diameter of hole: mm
<25	8–10
5–50	12
50–100	16
100–150	20
>150	25

Table 17 Suitable sizes of vent holes in tubular member

molten zinc, venting of hot gases and subsequent draining of zinc.

Sealed tubes which are to be galvanised will require vent holes to allow the escape of hot air during the hot-dipping process. **Table 17** indicates suitable hole sizes.

Sections are generally welded before galvanising and components bolted together after galvanising.

Metal spraying

An alternative method of applying a metallic coating to steelwork is by spraying. In this case, either zinc or aluminium can be used. The metal, in powder or wire form, is fed through a special spray-gun containing a heat source which can be either an oxy-gas flame or an electric-arc. Molten globules of the metal are blown by a compressed air jet onto the previously blast-cleaned surface. No alloying occurs and the coating, which consists of overlapping platelets of metal, is porous. These pores are subsequently sealed by applying a flood coat of clear or pigmented epoxy or polyurethane coating. Further painting is then optional.

The adhesion of sprayed metal coating to steel surfaces is considered to be essentially mechanical in nature. It is therefore necessary to apply the coating to a clean roughened surface; blast-cleaning with a course grit abrasive is normally specified.

Coating thickness varies between 100–250 μm for aluminium, and 75–400 μm for zinc. Both metals perform similarly in most situations, but aluminium is more durable in highly industrial environments.

Metal spray coatings are usually applied in the fabrication shop. Unlike hot-dip galvanising, there is no limitation on the size of the element and, as the steel surface remains cool, there are no distortion problems. Relative costs will vary depending on the size of the section, but metal spraying might typically be twice as expensive as galvanising.

The protection of structural steelwork against atmospheric corrosion by metal-sprayed aluminium or zinc coatings is covered by BS EN 22063.

Electroplating and sheradising

Electroplating and sheradising are processes which are used for the application of a metallic coating to small components such as fittings and fasteners. Due to the thin coating that is applied, they should generally only be used for mild environments, or for decorative purposes.

Common protection systems

The protection systems detailed in this section (see **Tables 18a and 18b**) are based on those which are currently recommended by Corus for various environments (A Corrosion Protection Grade for Steelwork Exposed to Atmospheric Environments). The recommended paint thicknesses are the dry film thickness. These thicknesses may vary between manufacturers. Detailed guidance on corrosive protection systems and tests are given in BS EN ISO 12944 and 14713.

Systems may be identified with respect to the following environments:

Interior

low risk: where there is a low risk of condensation, exhibition halls, workshops, sports halls.
medium risk: production buildings, or those with potentially high humidity and some risk of pollution.

Exterior

medium risk: most rural and urban areas.
high risk: urban and industrial atmospheres with moderate sulphur dioxide and/or coastal areas with low salinity.
very high risk: industrial areas with aggressive atmospheres. Coastal areas with high salinity.

The tables do not cover:

Interior

high risk: buildings with high humidity and corrosive atmospheres, e.g. chemical plants, swimming pools. In these cases, it is recommended that specialist advice is sought.

For steelwork in contact with a masonry outer skin, it is recommended that the steel is galvanised or, alternatively, it is blast-cleaned and one coat of solvent-free epoxy (dry film thickness of 450 μm) is applied. If the steelwork is galvanized, two coats of heavy-duty bitumen (total dry film thickness of 200 μm) should be applied.

Method and location of application

Spraying is the most widely adopted method for application of paint systems in a workshop with a controlled environment, often with forced ventilation. Both air-fed and airless spray are possible. Airless spraying is now more common, as application rates are higher and over-spray is reduced. Brush application is usually carried out on site, where control of the local environment is more difficult.

Site coating of previously primed steelwork is often preferred by steelwork contractors as it allows the steelwork to be moved rapidly out of the workshop and onto site. The other main advantage is that, as the final coat will be

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Environment	Interior – low risk			Interior medium risk					
	Hidden	Visible	Hidden	Visible	Hidden	Visible	Hidden	Visible	Visible
Structure life in years	60+	55+	Unknown	Unknown	60+	60+	55+	45	40+
Coating life in years[2]	N/A	15	N/A	Unknown	N/A	40+	N/A	25	20
Surface preparation	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5	–	–	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5
Shop applied coating	Zinc phosphate epoxy primer (80 μm)[3]	High-build zinc phosphate epoxy primer (80 μm)[3]	Water based acrylic or zinc phosphate epoxy primer (2 × 60 μm)[4]	Water based acrylic or zinc phosphate epoxy primer (2 × 60 μm)[4]	Hot-dip galvanise (85 μm)[5]	Hot-dip galvanise (85 μm)[5]	Zinc phosphate epoxy primer (80 μm) High build recoatable epoxy MIO (120 μm)	Zinc phosphate epoxy primer (80 μm) High build recoatable epoxy MIO (100 μm)	High solid epoxy zinc phosphate primer (100 μm)
Surface preparation	None	Wash free of contamination	None	Wash free of contamination	None	Mordant wash	None	Wash free of contamination	None
Surface coating	None	Alkyd finish (60 μm)	None	Water-based acrylic or epoxy finish (60 μm)	None	Vinyl primer (40 μm) Vinyl finish (60 μm)	Polyurethane finish (60 μm)	Polyurethane finish (60 μm)	High solid aliphatic polyurethane finish (120 μm)

Notes:
1 Structure life – the number of years of freedom of severe corrosion that might lead to weakening of the structure.
2 The expected number of years to first maintenance of the coating.
3 Can be applied as a 20 μm prefabrication primer plus 60 mm post-fabrication primer.
4 Water-based technology is still developing and advice should be obtained from the manufacturer.
5 A thickness of 85 μm can be achieved on steel over 6 mm thick without grit blasting.

Table 18a Recommended corrosion protection systems for interior environments

Environment	Exterior – medium risk			Exterior – high risk		Exterior – very high risk	
Coating life in years[1]	40+	20	20	20	25	20	20+
Surface preparation		Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5	Blast clean to SA 2.5
Shop applied coating	Hot-dip galvanise (85 μm)	Zinc phosphate epoxy primer (80 μm) High build epoxy MIO (100 μm)	High solid epoxy phosphate primer (100 μm) High solid aliphatic polyurethane finish (100 μm)	Zinc rich epoxy primer (140 μm) High build epoxy MIO (100 μm)	Zinc rich epoxy primer (140 μm) High build epoxy MIO (200 μm)	Zinc rich epoxy primer (140 μm) High build epoxy MIO (200 μm)	Zinc rich epoxy primer (140 μm) High build epoxy MIO (100 μm)
Surface preparation	None	Wash free of contamination	None	Wash free of contamination	Wash free of contamination	Wash free of contamination	Wash free of contamination
Surface coating	None	Recoatable polyurethane finish (60 μm)	None	High build epoxy MIO (100 μm)[3]	High solid aliphatic polyurethane finish (80 μm)	High solid aliphatic polyurethane finish (80 pm)	Recoatable polyurethane finish (60 μm)

Notes:
1 Coating system durability is based on experience and is the expected life in years before major maintenance. It is not a guaranteed life expectancy.
2 A thickness of 85 μm can be achieved on steel over 6 mm thick without grit blasting.
3 It should be noted that the available colours are limited.

Table 18b Recommended corrosion protection systems for exterior environments

applied on site, repair of damage caused by transport and erection is not required and the finished appearance will be more uniform.

However, a number of important aspects should be considered, as noted earlier:

- The quality of the site-applied finish coat will be inferior to that applied in the workshop, particularly if the atmosphere is polluted or in coastal regions.
- All paintwork should be cleaned on site, before application of the site coat.
- On-site coatings should generally be applied by brush.
- Damage to paint systems should be minimised by the careful use of strops, wrapped chains or lifting lugs.

Protection of connections

The method of protecting the connections depends on the type of connection and the system used for the steelwork. In severe environments, galvanised bolts may be used.

Normal bearing-type bolted connections require protection of the contact surfaces. Although a priming coat is sometimes recommended, it is worthwhile to treat the contact surface to at least the same standard as the main steelwork, as it is unlikely that these surfaces will be repainted. In extreme cases and over a long period, corrosion between the surfaces can lead to lamination of the steel and failure of the connection.

For high-strength friction grip (HSFG) bolted connections, the contact surface should be free from any containment or coating which would reduce the slip factor required in the connection. Most paint systems will reduce the frictional resistance, but some inorganic zinc silicate primers and spray metal coatings can increase the slip resistance of the connection. Where required, the contact surfaces should be masked to prevent painting of the surfaces. All HSFG connections should be sealed round their edges when the bolts have been tightened. After making the connection, the area should be painted to repair any damage caused to the paint system, and to protect the exposed part of the connection.

Fire protection

Fire resistance is a requirement of all national Building Regulations in order to ensure stability of the structure in the event of fire, both to allow means of escape of the occupants and also for the safety of fire fighters. Although not stated explicitly in the Regulations, fire resistance measures also ensure protection against disproportionate damage, so that small fires will not require major structural repair. Fire resistance requirements are a function of:

- the size of the building (especially the height)
- the fire load (combustible contents)
- presence of 'active' fire protection measures, e.g. sprinklers.

A holistic 'fire engineering' approach can be used to predict the severity of fires, and their effect on the primary structure. Fire engineering is often used to justify the use of unprotected steel structures in airport terminals and sports halls etc, where fire loads are low and means of escape are good.

The following sections present the strategies that may be employed to ensure the fire safety of steel structures, depending on whether the steelwork is concealed (i.e. is not visually important), or is exposed to view. Fire tests are now specified to European test standards BS EN 1365, which differ in some specific details from BS 476, and so BS EN test results may be different (specify lower fire resistance periods) than to the former BS.

Forms of fire protection

SCI and the Association of Specialist Fire Protection Contractors and Manufacturers Ltd (ASFP), have produced detailed guidanc;e listing the majority of approved materials and presenting tables of protection thicknesses for various periods of fire resistance.

In principle, the main forms of fire protection to steel members that may be considered, are as follows:

- Sprayed protection around the profile of the member. Sprayed materials may be of a variety of forms, including, for example, cementitious materials and vermiculite-cement.
- Board protection as a box around the member. Board protection can be fixed on secondary noggins, or in some cases, to itself to form the box. Board protection is preferred for columns, because it forms the finished shape of the column. Softer forms of protection, based on mineral fibre, are also used for beams and trusses.
- Wrapping in fire protective layer and sheathing by pressed metal panels to preserve the architectural profile of the member.
- Intumescent coatings, which expand on heating to provide an insulated layer. Essentially these coatings have the appearance of paint, but thicker coatings can have a fibrous appearance.
- Concrete encasement of I sections. Partial encasement between the flanges of I sections can achieve good fire resistance, and can be introduced as an off-site process.
- Concrete-filling of tubular columns, including provision of additional reinforcement for larger periods of fire resistance.
- Water-filling of tubular sections, as part of a continuous tubular structure, such as a roof truss or an external structure.

Fire protection systems are generally assessed on a failure temperature of 550°C for columns, which corresponds to approximately 50% loss of strength of the steel member, or 610°C for beams. Some manufacturers present design tables for a range of failure temperatures, which may be useful when selecting the fire protection required for lightly stressed members.

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Sprayed and board protection

Sprayed protection is applied manually by spraying around the profile of the section, which results in a rough surface texture. Therefore, sprays are generally used in concealed applications, such as beams with suspended ceilings. A typical sprayed protection to cellular beam is shown in **Figure 4**. For most applications, a spray thickness of 20–30 mm would be suitable depending on the resistance period.

Board protection is of two types: hard boards, and softer boards. Hard boards, such as fire-resistant plasterboard, can be used as a fascia, whereas softer boards are usually concealed. Boards provide a 'box' protection around the member, and are held in place by a light framework. The enclosed box is usually 70–100 mm wider than the steel section.

Intumescent coatings

Intumescent coatings are applied as paints. They expand on heating to form an insulating layer which protects the steel section. There are two basic types of these coatings:

- Thin film coatings (<2 mm thick). These coatings can generally achieve 90 minutes fire resistance for beams and columns. They are generally appropriate for internal applications.
- Thick film coatings (3–6 mm). These coatings can provide up to 120 minutes fire resistance (or longer in some cases). They are often appropriate for external applications.

Thin film coatings are a particular type of coating and not a thinner film of a thicker system. Thin film coatings retain the basic shape of the steel section, whereas thick film coatings can appear relatively coarse, and may have an 'orange peel' texture. Finishing coats may or may not be applied on site. Some fabricators prefer on-site application rather than factory-applied coatings, as handling on site may cause some damage. Furthermore, touching up of the surface does not achieve the same quality of finish as a factory-applied coating.

Off-site application of intumescent coatings are widely used because of the implications on a reduced construction programme. Guidance on the use of off-site intumescent coatings is given in SCI publication (P-160). They are applied as a single coating of up to 1.5 mm thickness in order to speed up the drying operation.

The exposed area of tubular sections is considerably less than that of open sections, such as I beams. Therefore, tubular sections heat up less quickly in a fire, and so require less fire protection to achieve a specific fire resistance. However, SHS or RHS sections require a slightly greater coating thickness than CHS sections because of possible cracking of the coating at the corners of the section.

Partial encasement by concrete

Many forms of steel construction have been developed, such as slim floor beams and partially encased columns, which do not require additional fire protection.

In slim floor beams, only the bottom of the beam flange or plate is exposed, and the remaining part of the steel section is encased by the concrete or the slab. In fire conditions, the upper part of the section resists the applied moment, despite the much reduced strength of the bottom flange. A range of asymmetric steel ASB sections has been designed specifically to achieve 60 minutes fire resistance. Additional protection may be applied to the bottom flange for longer fire resistance periods.

Partially encased columns are formed by placing concrete between the flanges (usually as an off-site operation). Bar and link reinforcement may be attached to the steel section. In fire conditions, the flanges of the column (or beam) are exposed but the steel web, concrete and bar reinforcement remain relatively cool and resist the applied compression. A fire resistance of 60 or 90 minutes can be achieved, depending on the amount of reinforcement provided.

Fully encased columns are rarely used in modern steel construction, except where durability or resistance to impact is a concern, or long periods of fire resistance are required. The size of the encased column is usually 150–200 mm wider than that of the steel section.

Concrete filling of tubular sections

Tubular steel columns can be filled with concrete to provide up to 120 minutes fire resistance with additional bar reinforcement. Concrete filling is carried out on site and so the fast erection times of steel construction are maintained.

Concrete filling of tubular sections requires no special equipment and the filling operation may be integrated into other concreting operations. However, it is necessary to provide ventilation holes in the column walls to prevent the build-up of steam pressure in the event of a fire. Two 10 mm diameter holes placed diametrically opposite each other both at the top and bottom of each storey height have proved to be adequate. A drain hole should also be provided at the base of a column to prevent water collecting if it is left standing empty on site prior to filling.

For large-diameter tubular columns, concrete may be pumped from the base of the column through a valve, which is later cut away when the concrete has gained adequate strength. Two or three storeys may be filled in this method.

Considerable research has been undertaken into the structural and fire-resistance performance of concrete-filled hollow sections, aimed at developing design procedures for this form of construction. The results of recent research in the subject is incorporated into national and international codes of practice. Calculation methods are included in BS EN 1994-1-2: Eurocode 4 Part 1.2.

Water filling of tubular sections

Water filling is often mentioned as a method of providing fire resistance to tubular structures, but is rarely carried

out in practice. The most well-known example is the external structure to a building in Cannon Street, London. At the cargo handling facility of Hong Kong airport, the long-span tubular trusses were water filled to provide 120 minutes fire resistance, and the prototype trusses were fire tested to prove their effective performance. The prerequisites of this system are a continuous passage for water through the tubular structure, which is itself problematical in structural design terms, and the need for a water replenishment system, which is fed by gravity.

Site installation

The following sections refer to practical aspects of steel construction, including bolting, welding, deflections and allowance for tolerances. More general guidance is given in the BCSA National Structural Steelwork Specification for Building Construction (NSSS).

Bolting

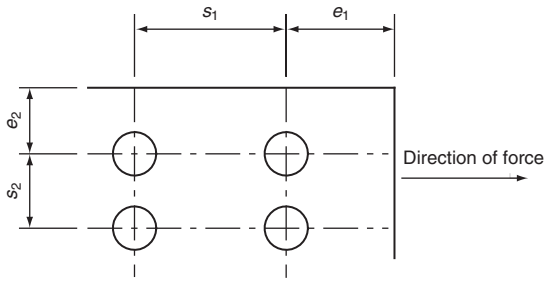
A limited range of bolt diameters and lengths is usually detailed on a given project, depending on the plate or element thickness to be joined. Fully threaded bolts are gaining popularity as they can be used with a range of plate or member thicknesses.

Countersunk holes and bolts are often appropriate for use on the outside faces of columns or beams to avoid protrusion of the bolt heads, although fabrication using countersunk bolts is more expensive. In certain cases, countersunk bolts may also reduce the achievable resistance of the connection.

Structural bolts fall into two groups: 'ordinary' or 'black' bolts, and preloaded or high-strength friction grip (HSFG) bolts. All bolts are available in a range of lengths, available generally in 10 mm increments, and various diameters. The industry standards for general application are 20 and 24 mm diameter grade 8.8 bolts. Fully threaded bolts are produced in a small range of lengths. For M20 grade 8.8 bolts, the preferred length of bolt is 60 mm.

Preloaded or HSFG bolts are generally only used in applications where bolt slip would otherwise impair the structural performance, or where the structure is exposed to dynamic or fatigue loading. HSFG bolts are produced from the same material as grade 8.8 bolts but have a larger head and stronger nut to avoid tensile failure during tightening by a torque wrench. Because a connection using HSFG bolts relies on friction between the steel interfaces rather than the bearing capacity of the bolt, a connection using HSFG bolts may require marginally more bolts than a similar connection using grade 8.8 bolts. The higher cost of HSFG bolts and their tightening procedure means that they are more expensive than grade 8.8 bolts.

Minimum and maximum bolt spacings and edge distances are specified in BS 5950-1 and are summarised in **Figure 4**. Maximum edge distances are introduced only where there is a risk of corrosion between the plates.



	Min.	Max.	
e_1	1.4D	11t or (40 mm + 4t)*	D = hole diameter
e_2	1.4D	11t or (40 mm + 4t)*	d = bolt diameter
s_1	2.5d	14t or 200 mm*	t = thickness of thinner plate
s_2	2.5d	16t or 200 mm*	* = when subject to possible corrosion

Figure 4 Minimum and maximum bolt spacings and edge distances in connections subject to shear forces

Standards for bolts

Since the 1970s, the standards for bolts used in the UK have remained largely unchanged although the steelwork design standards in that time have evolved. A new generation of BS EN standards for bolts is listed in the BCSA's National Structural Steelwork Specification (BCA, 2007), and includes grade 8.8 non-preloaded bolts to BS EN ISO 4014 and 4017, and nuts to BS EN 4032. However, the use of such BS EN ISO bolts is rare as they are more expensive than the equivalent bolts to BS 4190.

Non-preloaded bolts to EN 15048

There are many more types or variations of non-preloaded bolts across Europe than preloaded bolts, although their technical differences are not as large as that of preloaded bolts. Therefore a much simpler product standard is being developed for non-preloaded bolts than for preloaded bolts. EN 15048 has the general title: Non-preloaded Structural Bolting Assemblies, and is divided into two parts, as follows:

- EN 15048-1: General Requirements
- EN 15048-2: Suitability Test

Strength grades from 4.6 to 10.9 are included. In the UK, bolts to BS 4190, a slightly deeper nut, which should conform to the EN 15048 suitability test.

Preloaded bolts

Across Europe, two different philosophies exist to achieve the necessary ductility in preloaded bolt assemblies. Both systems have long histories and are well proved.

The approach in BS 4395 is to use deep nuts and longer thread lengths in the bolt assemblies and thus obtain ductility predominantly by plastic elongation of the bolt. The

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longer threaded length is necessary to ensure that the induced strain is not too localised. In load tests, these bolts tend to fall in a ductile manner by rupture in the shank of the bolt at the root of the thread. These bolts are generally insensitive to over-tightening during preloading and thus require less site control. BS 4395 bolts only require plain hardened washers.

The approach in Germany, for example is to use shallower nuts and shorter thread lengths in the bolt assemblies and thus obtain ductility by plastic deformation of the engaged threads rather than in the threaded shank zone outside of the engaged length. Although the failure mode of the bolt assembly is brittle, it is argued that it is a more ductile failure mode for the steelwork connection because the bolt shank remains in place after failure and may act as a 'peg'. These bolts are generally more sensitive to over-tightening during preloading and thus require more site control. These bolts have an under head radius and therefore require a washer with an internal chamfer (and for this to be installed in the correct orientation if only chamfered one side). It is quite common to use DIN 6914 bolts in non-preloaded situations.

Therefore, in developing the European product standard EN 14399: High-strength Structural Bolting Assemblies for Preloading, it was agreed to develop two general parallel systems, HR (British/French) and HV (German), which reflect the above two philosophies. Both systems have under head radius and thus require chamfered washers to be placed under the heads of the bolts. The HR system will have two grades, 8.8 and 10.9, while the HV system will only have one grade, 10.9 t. It is important to avoid mixing up the components of both systems. Therefore bolts and nuts for both systems are standardised in separate parts of the product standard EN 14399 and the marking of the same system is uniform.

EN 1090-2 permits the selection of a tightening method from the three methods outlined below unless otherwise specified.

Torque control method

In this method the torque is to be applied in two steps. The first step is to apply a torque of up to 75% of the required value to all the bolts. The second step is to apply an additional torque to each bolt such that the torque applied to a bolt is up to 110% of the required nominal torque value. The extra 10% is to offset the subsequent preloading force decrease.

Combined method

This method is a combination of torque control and the traditional 'part-turn' method. After the joint is bedded, preloading takes place in two steps. The first step is to apply a torque of up to 75% of the required torque value to all bolts (as with the torque control method). The second step is to apply to each bolt a predetermined rotation of 'part-turn' to a specified angle, depending on the bolt length.

As an alternative to using the specified angles, the angle of rotation may alternatively be determined by test. It should be noted that the simple part-turn method used in the UK for many years will no longer be permitted in EN 1090-2 due to the risk of over-tightening bolts, which is the same reason that the simple part-turn method has not been allowed for Part 2 higher-grade HSFG bolts to BS 4395.

Direct tension indicator method

This method relies on protrusions on special washers often called load-indicating washers. These protrusions create a gap prior to preloading in the installed assembly. After the joint is bedded down, on tightening the bolt, the gap reduces as the protrusions depress under load. When the actual gap is less than the maximum specified gap, the bolt force will not be less than the specified preload.

Details of the procedure test for determining the slip factor in connection with preloaded bolts are found in Annex G of EN 1090-2. It is more complex than the traditional British approach in that it requires five tests instead of three, and slip is measured at a higher value of 0.15 mm instead of 0.10 mm. The slip load is averaged over the five tests.

The emergence in the last 10 years of the tension control bolt (TCB) as a preloaded bolt has been the one major development in the UK market. The modern TCB has been introduced into the UK from versions currently used in the USA and Japan.

Typically the TC bolt head is round but may be hexagonal. Preloading is normally carried out by an electrical shear wrench at the threaded end of the bolt (i.e. the nut or spline end) and preloading is complete when the spline shears off. TC bolts are distinctive.

Welding

Most fabrication shops use manual metal arc (MMA), semi-automatic and fully automatic equipment, depending on the weld type and length of run. The detailing of the steelwork must take account of the welding procedure. The welding processes that are commonly used are shown in **Table 19**.

Welding equipment has become much lighter, and more readily portable and easier to use in site applications. Normal manual metal arc welding (MMA) techniques are usually employed for site welding, although preheat may be required for some types and thicknesses of steel. Overhand or downhand welding is preferred because of the difficulty (and hence the slow speed) of underhand welding. Situations with underhand site welding should be eliminated by careful design.

Two generic forms of welded connection between steel plates and steel sections are butt and fillet welds.

Workshop or site	Comments	Maximum size fillet weld in single run	Process	Automatic or manual	Shielding	Main use
Workshop or site	Fillet welds larger than 6 mm are usually multi-pass, and are more expensive than single-pass	6 mm	Manual metal arc (MMA)	Manual	Flux coating on electrode	Short runs
Workshop or occasionally on site	With twin heads, simultaneous welds either side of joint are possible	10 mm	Submerged arc (SUBARC)	Automatic	Powder flux deposited over arc and recycled	Long runs or heavy butt welds
Workshop	MIG has replaced manual welding in many workshops	8 mm	Metal inert gas (MIG)	Automatic or semi-automatic	Gas (generally carbon dioxide – CO_2)	Short or long runs

Table 19 Commonly used welding processes

Fillet welds are preferred in general construction because of their ease of placement and their lower heat input. For welded plates, symmetrically placed fillet welds reduce angular distortion. Single-sided fillet welds are usually only used in connections between hollow sections. Butt welds require preparation of the ends of the elements to be welded and are therefore more expensive. However, they can achieve full strength of the connected elements.

Welds always have a characteristic rippled surface. Sometimes it is requested that welds should be ground down to a smooth finish for aesthetic or structural reasons. Welds suitable for grinding down are made a little larger, as metal is lost in grinding and the strength has to be maintained. Specifications often state 'All welds to be ground down'. It may be possible to make worthwhile savings by specifying 'All visible welds to be ground down', unless there are structural reasons for removing surface defects, for example in a fatigue-sensitive connection.

Fillet welds are generally used to attach end plates or brackets. Fillet welding is usually cheaper than butt welding but it relies on projections (of up to 15 mm) to be able to adequately place the weld.

A fillet weld can be specified by its throat thickness and/or leg length. Where the throat thickness is not specified, the actual throat thickness should not be less than 0.7 times the specified leg length for concave fillet welds, and not more than 0.9 times the actual leg length for convex fillet welds.

Fillet and fillet-butt welds in tubular members

Fillet-butt welds are used when welding CHS or RHS members as the ends of the incoming section may be profiled. The terms describe the welding conditions which apply when various size ratios of bracing to main member are involved.

Figure 5 shows the weld details for two bracings meeting main members of considerably different sizes. In both cases, welding conditions at the crown are similar and so for the same loads, identical fillets would be used. However, conditions differ at the side of the tube. The curvature of the larger main member gives good conditions for fillet welds, while the curvature of the smaller main member necessitates a butt weld. The change from fillet to butt welding must be continuous and smooth.

For calculating weld sizes, both types are considered as fillet welds. The fillet-butt preparation is used where the diameter of the bracing is one third or more of the diameter of the main member.

Continuous welding is preferred when connecting SHS or RHS members, even if not required structurally. This is advised because of the possible risk of corrosion in the gaps between the elements.

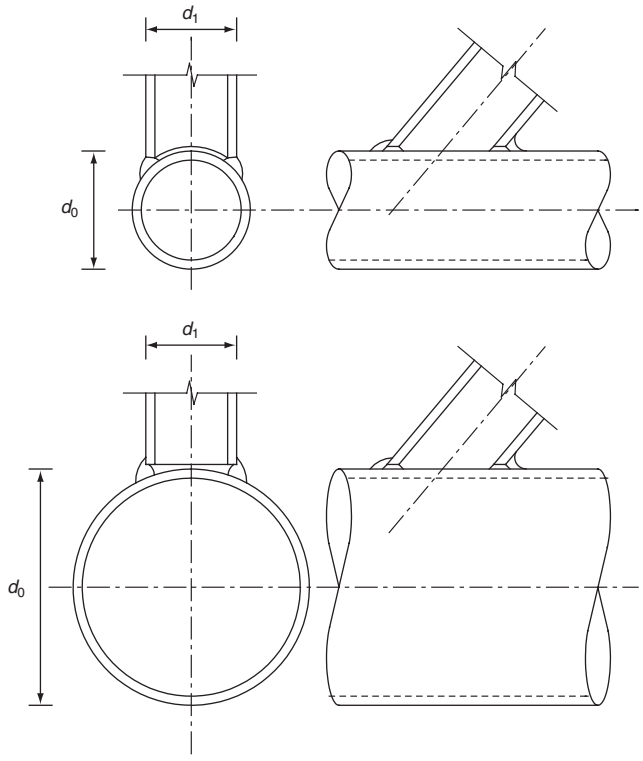

Figure 5 Fillet and fillet-butt welds to CHS members

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Welding conditions

Welding should be carried out in workshops or under controlled conditions. Precautions should be taken during site welding to protect the member from adverse weather conditions. When surfaces to be welded are wet, or where the ambient temperature is less than 0°C, the sections must be heated locally until they are warm to the touch for a distance of not less than 75 mm on either side of the joint. Furthermore, the surfaces to be welded should be dry, clean and free from rust, oil, grease and paint (except 'weld through' primer).

When subjected to elevated temperatures during welding or cutting, fumes will be produced. The primer paint should be removed in the area which is to be treated, or provision should be made for adequate ventilation during welding or cutting which, if necessary, should include local fume extraction.

Tolerances

The National Structural Steelwork Specification for Building Construction (NSSS) published by BCSA is the industry standard for specifiers and fabricators. It provides greater uniformity in specifications issued with tender and contract documents. Appropriate tolerances should be agreed by all the parties at the start of the project, as late changes to allow for these effects can be costly.

Deviations can be dealt with by two means: providing adjustment to overcome local lack of fit (e.g. shims), or providing clearance to give two adjacent components separate zones. Both adjustment and clearance provisions are often referred to as tolerances.

The tolerances provided are permitted deviations set by the designer to ensure consistency with the design assumptions. Two generic types of tolerance exist:

- Geometric variations in section size and also in the fabricated members.
- Member deviations after completion of the erected structure.

Limitations on these two forms of tolerance are presented in **Tables 20** and **21**. Stricter tolerance limits may be appropriate, but this may lead to increased costs of the structure. Special considerations may be required for glazing supports. The basic dimensional tolerances for rolled sections are given in the NSSS.

Recently, construction tolerances are specified in BS EN 1090-2, which defines 'essential' and 'functional' requirements for steel structures. These tolerances differ slightly from the NSSS limits and should be used when designing steelwork to Eurocode 3 and 4.

Criterion for tolerance	Tolerance for section type	
	Rolled UB/UC	Fabricated section
Depth of section, D: (mm)		
D < 180	+3.0 or −2.0 mm	±4.0 mm
180 < D < 400	+4.0 or −2.0 mm	
400 < D < 700	+5.0 or −2.0 mm	
D > 700	+6.0 or −2.0 mm	
Width of section, B: (mm)		
B < 110	+4.0 or −1.0 mm	4.0 mm
110 < B < 210	+4.0 or −2.0 mm	
210 < B < 325	+4.0 or −4.0 mm	
B > 325	+6.0 or −5.0 mm	
Off-centre of web		
102 < D < 305	3.2 mm	±5.0 mm
D > 305	4.8 mm	
Bow of web	–	Greater of d/150 or 3 mm
Horizontality of flange		
B < 110	1.5 mm	Greater of B/100 or 3 mm
B > 700	2% of b ⩾ 6.5 mm	
Verticality of web at support	–	Greater of d/300 or 3 mm
Squareness of cut end not prepared for bearing (plan or elevation)	D/300	–
Squareness of cut end prepared for bearing (plan or elevation)	D/1000	–
Overall length	±2 mm	±3 mm
Straightness along length of beam	Greater of L/1000 or 3 mm	Greater of L/1000 or 3 mm

Notes:
L is member length; d is web depth; D is section depth; B is the section width (all in mm)

Table 20 Typical geometric tolerances for steel sections (adapted from BCSA National Structural Steelwork Specification)

Deflections

Control of deflections and long-term movement is important if other elements that are attached to the structure may be adversely affected, or if the use of the building is impaired, such as by water leakage. It is not appropriate here to discuss the wide range of deflection limits that are specified for different applications, but to note certain well-defined cases that the designer should address, as follows:

- Edge beams supporting cladding, such as glazed façades.
- Sway deflection of unbraced frames subject to wind action.

Level of adjacent beams	±5 mm
Position of floor beams at columns	±10 mm
Level at each end of same beam	5 mm
Position of beam from wall	±25 mm
Level of foundations	+0 to −3 mm
Position of holding-down bolts	20 mm
Maximum gap between bearing surface of column end (depth D)	D/1000 + 1 mm

Table 21 Acceptable tolerances in general steel construction (adapted from BCSA National Structural Steelwork Specification)

- Vertical movement of low-pitch portal frames or roofs.
- Deflection of beams supporting internal compartment walls.
- Visual deflections or installation of raised floors in long-span beams.
- Horizontal movement of tall structures.
- Use of isolating pads for deflection-sensitive equipment.
- Movement of supports to cranes and travelling machinery.

The designer should agree sensible deflection limits with specialist suppliers of the cladding and lifts etc., and with the steel fabricator. The normal limit on deflections for beams subject to imposed load is beam span/360, in order that deflections are not noticeable and that partitions are not subject to cracking. Stricter deflection limits may be required in many of the above cases. A limit of span/500 is often specified for edge beams supporting brittle forms of cladding. It may be necessary to pre-camber long-span beams (open >12 m) in order to offset permanent deflections which may be visually unacceptable. The amount of pre-camber is normally set at half the anticipated total deflection, but not less than a practical minimum of 25 mm.

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Section 6: Polymers

Section editor: Christopher Hall

Chapter 46

Polymeric materials: an introduction

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The polymers are a diverse group of engineering materials. They are the main components of plastics, rubbers, resins, adhesives and paints. These materials have distinctive microstuctures built from macromolecular chains and networks of carbon and other light elements.

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Introduction

Most polymer materials are of organic composition, that is they contain carbon, and are composed of large molecules (macromolecules) each built of many atoms. They include materials such as polyethylene, poly(vinyl chloride), polyamide and epoxy resins. A list of polymers used in engineering along with their standard abbreviations is presented in **Tables 1** and **2**. These abbreviations are helpful to engineers since they simplify the complicated and unfamiliar chemical names of polymers. For a more detailed description and discussion of the molecular structures, classifications and nomenclature of polymers there is an abundance of specialised literature (for example, Hall, 1989, Painter and Coleman 1997, Osswald and Menges, 2003).

Polymers are nature's materials of choice: proteins, DNA and polysaccharides are macromolecules. Natural polymers generally have well-defined, precise and highly complex molecular structures. In engineering, most of the polymers used have simpler structures and are produced synthetically.

Definitions, structure and types

The molecular structure of polymers is responsible for many of the intriguing physical properties which lie behind their various applications. Polymers are composed of very large molecules (*macromolecules*) which consist of smaller units, called monomers, tightly bonded together with (strong) covalent bonds, as shown schematically in **Figure 1** for the case of a linear polymer chain. The chemical formula is of the type $–(A)_n–$ where A represents the monomer and the integer number n, called the *degree of polymerisation* or *polymerisation index*, is the number of monomers composing the chain. The length of the polymer chain (and the molar mass) is proportional to n.

Polymer architecture at the molecular scale can be rather diverse. In **Figure 2** three possible molecule architectures are depicted, leaving out the atomic scale chemical details and representing them with lines. This is a useful representation of the molecular conformations of polymers in space and is used extensively in demonstrating their microstructure.

A linear polymer consists of a long linear chain of monomers. A branched polymer comprises a long backbone chain with several short side-chain branches covalently attached. Cross-linked polymers have monomers of one long or short chain covalently bonded with monomers of another short or long chain. Cross-linking results in a three-dimensional molecular network; the whole polymer is a giant macromolecule.

Another useful classification of polymers is based on the chemical type of the monomers (**Figure 3**): *homopolymers* consist of monomers of the same type; *copolymers* have different repeating units. Furthermore, depending on the arrangement of the types of monomers in the polymer chain, we have the following classification: the different repeating units are distributed randomly (*random copolymer*) or there are alternating sequences of the different monomers (*alternating copolymers*) in *block copolymers* long sequences of one monomer type are followed by long sequences of another type; and *graft copolymers* consist of a chain made from one type of monomer with branches of another type.

Elastomers (also called *rubbers*) are lightly cross-linked networks while *thermosets* are densely cross-linked networks. Thermosets soften mildly and ultimately degrade upon heating, while thermoplastics, which do not contain cross-links, melt upon heating and they can be reshaped repeatedly. These thermomechanical differences between polymers, owing to the significantly different organisation at the molecular scale, have important consequences both in their processing and usage. Rubbers are characterised by the property of high elasticity, i.e. elastic behaviour at high stresses and strains. Polymers can be diluted in a variety of solvents (usually organic but there are a few polymers called *polyelectrolytes* which are water soluble). A sufficiently dense polymer solution can be cross-linked to form a polymer *gel* which is a *soft solid*.

Polymer materials may also be formed by blending two or more polymers into physical mixtures. For example, the rather poor impact strength of polystyrene is greatly improved by incorporating small particles of an elastomer. This material, *high impact polystyrene* (*HIPS*), was an early example of a polymer hybrid or alloy (**Figure 4**). Such polymer alloys show a distinct two-phase microstructure, often with chemical grafting at the interface. Another

Polymer	ISO abbreviation	Other customary names
Homopolmers		
Polyethylene	PE	Polythene
Polypropylene	PP	
Polystyrene	PS	
Polybutylene	PB	
Poly(methyl methacrylate)	PMMA	Acrylic, perspex, plexiglas
Polytetrafluoroethylene	PTFE	Teflon
Poly(vinyl fluoride)	PVF	
Poly(vinylidene fluoride)	PVDF	
Poly(vinyl chloride)	PVC	Vinyl
Poly(vinylidene chloride)	PVDC	
Poly(vinyl acetate)	PVAC	
Poly(vinyl butyral)	PVB	
Poly(ethylene terephthalate)	PETP	
Polyetheretherketone	PEEK	
Polyacrylonitrile	PAN	
Polyethersulphone	PESU	
Polycarbonate	PC	
Poly(butylene terephthalate)	PBTP	
Polyoxymethylene	POM	Acetal, polyacetal
Polyamide	PA	Nylon
Polyacrylamide	–	
Poly(phenylene oxide)	PPO	
Poly(phenylene sulphide)	PPS	
Epoxy	EP	Epoxide
Polyurethane	PUR	
Natural rubber	NR	
Polyisoprene rubber (synthetic)	IR	
Polychloroprene rubber	CR	Neoprene
Silicone polymers	SI	Polysiloxanes
Copolymers, hybrids and alloys		
Acrylonitrile-butadiene-styrene	ABS	
Melamine-formaldehyde	MF	
Phenol-formaldehyde	PF	
Urea-formaldehyde	UF	
Unsaturated polyester	UP	
Styrene-acrylonitrile	SAN	

Table 1 Some engineering polymers

important rubber-toughened commodity polymer, acrylonitrile-butadiene-styrene (ABS), has particles of styrene-butadiene rubber dispersed in a glassy styrene-acrylonitrile (SAN) matrix. Other well-established polymer hybrids (also called *blends*) include polyphenylene oxide/polystyrene (PPO/PS) and poly(methyl methacrylate)/poly(vinyl chloride) (PMMA/PVC).

Polymer	ISO abbreviation	Other customary names
M group		
Chloropolyethylene rubber	CM	
Chlorosulfonylpolyethylene rubber	CSM	
Ethylene-propylene-diene rubber	EPDM	
O group		
Epichlorohydrin rubber	CO	Epoxide rubbers
Q group		
Fluorosilicone rubbers	FVMQ	
Silicone rubber	MQ	
R group		
Acrylonitrile-butadiene rubber	NBR	Nitrile rubber, Buna-N
Butadiene rubber	BR	
Chloroprene rubber	CR	Neoprene, polychloroprene
Isobutene-isoprene rubber	IIR	Butyl rubber
Isoprene rubber	IR	Polyisoprene
Acrylonitrile-isoprene rubber	NIR	Nitrile rubber
Natural rubber	NR	
Styrene-butadiene rubber	SBR	
T group		
Polysulphide rubbers	OT, EOT	
U group		
Polyester urethane	AU	Polyurethane
Polyether urethane	EU	Polyurethane

Table 2 Some engineering polymers: elastomers

Many properties of polymeric materials depend on the microscopic arrangement of their molecules. Polymers can have an amorphous (disordered) or semicrystalline

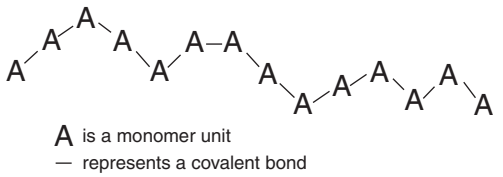

Figure 1 A polymer chain

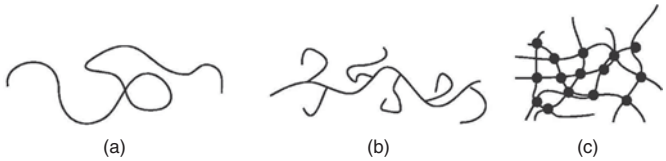

Figure 2 Types of molecular architectures: (a) linear chain; (b) branched molecule; (c) cross-linked network; molecules are linked through covalent bonds, the network extends over the whole sample forming a giant macromolecule

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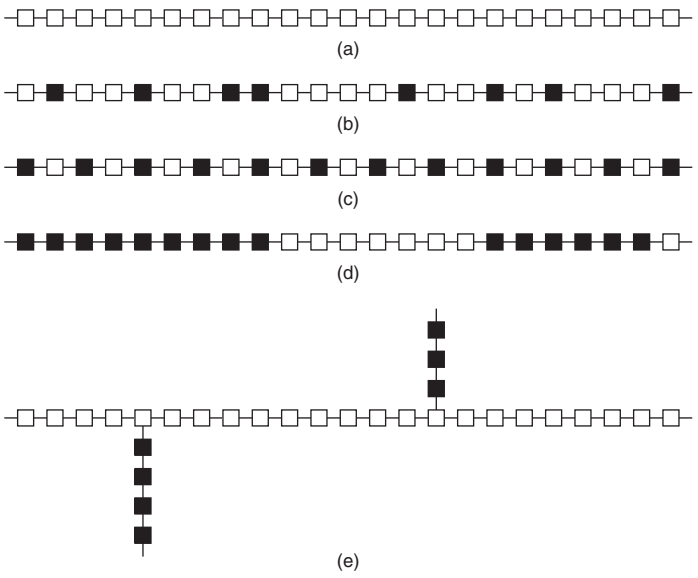

Figure 3 (a) Homopolymer; (b) random copolymer; (c) alternating copolymer; (d) block copolymer; (e) graft copolymer

(partially crystalline, partially ordered) structure (**Figure 5**). Amorphous polymers lack order and are arranged in a random manner, while semicrystalline polymers are partially organised in regular crystalline structures.

Polymers are often mixed with inorganic particles (usually in the form of continuous fibres, such as glass or particulates such as mica, talc and clay) in order to modify and improve (mainly but not exclusively) their mechanical properties. Reinforcement using organic fibres (for example, kevlar (poly(paraphenylene terephthalamide)

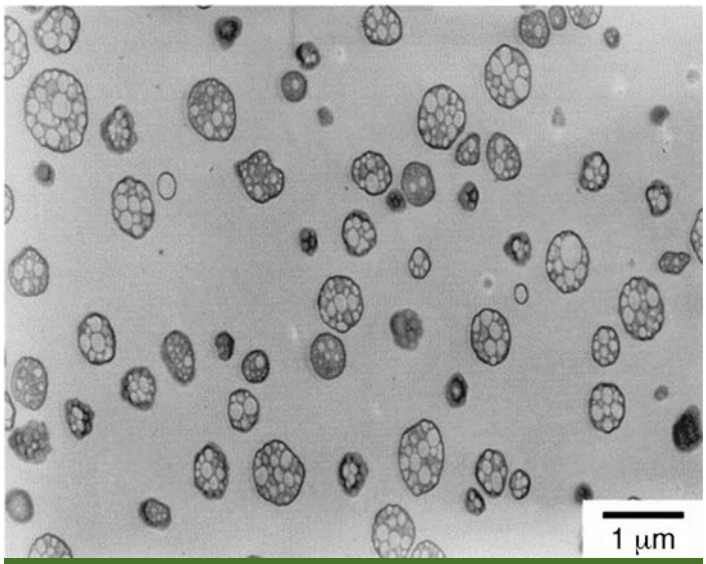

Figure 4 Microstructure of high-impact polystyrene: rubber particles within a polystyrene matrix (reprinted from T. Kuboki *et al*., Macromolecules ©2002 American Chemical Society)

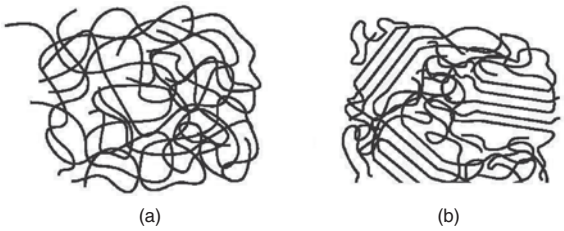

Figure 5 (a) Amorphous polymer (observe the entanglements among the polymer chains) and (b) simplified model of a semicrystalline polymer (observe the crystalline and amorphous domains)

or *aramid*) or carbon fibres) is also possible. Such *composite* materials are fully described elsewhere.

There is another way of classifying polymers according to their application areas: (1) plastics (for structural components, packaging); (2) elastomers (for damping or high friction); (3) fibres (for reinforcement); (4) coatings (for protection of materials surfaces); and (5) adhesives (for joining of structural components).

Specific examples and chemical structure

Polyethylene (PE) has the simplest molecular structure which is shown in **Figure 6**. This structure corresponds to a chemical formula of the form $-(CH_2-CH_2)_n-$. The monomer unit is shown within the parenthesis and reveals that PE is produced from ethylene gas, $CH_2=CH_2$, by breaking the double covalent bonds and connecting the gas molecules consecutively at high pressure. For PE, n can take values from a few hundred to hundreds of thousands. It has to be noted that the occurrence of side chains or branching (**Figure 2(b)**) is inevitable if PE is produced by way of the high-pressure polymerisation route resulting in low-density polyethylene (LDPE). This has important consequences for the polymer microstructure and physical properties. It can be avoided by employing catalysts and low-pressure synthesis; in this case, the branching is minimal and high-density polyethylene (HDPE) is attained. Higher density allows a closer packing of PE chains at the molecular scale which promotes crystallisation and higher stiffness and strength. Some typical applications of LDPE are in packaging and insulation for

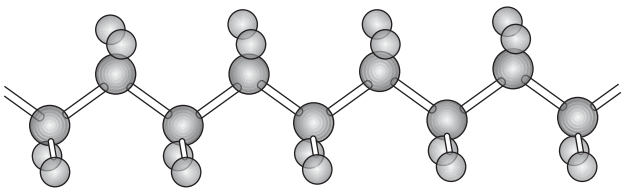

Figure 6 Molecular structure of linear polyethylene depicting the carbon atoms of the chain backbone covalently bonded in a linear fashion. Each carbon atom is also covalently linked to two hydrogen atoms

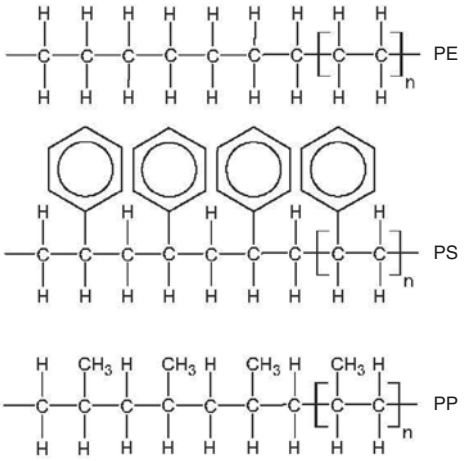

Figure 7 Some major hydrocarbon polymers

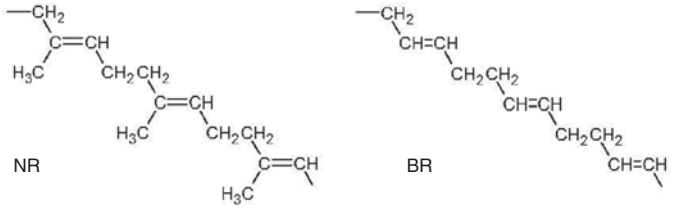

Figure 8 Some major hydrocarbon polymers: rubbers

electrical cables, while HDPE is used for heavy-use containers such as tanks, pipes and structural panels.

The molecular chain structures of many commercial polymers are shown in **Figures 7, 8 and 9**. Poly(vinyl chloride) (PVC) and polypropylene (PP) have molecular structures similar to that of PE, based on –C–C– chains with the important difference that one hydrogen atom in the monomer is substituted by the element chlorine, Cl, in PVC, and by the methyl group, CH_3, in PP. In the case of polytetrafluoroethylene (PTFE) all hydrogen atoms are replaced by the element fluorine, F. PP is used widely in water and gas pipes, PVC in pipe, ducting and window frames while PTFE's low surface energy finds many applications in low friction coefficient applications such as bearings. Polystyrene (PS) has the same basic backbone but one hydrogen is substituted by a bulky aromatic (or benzene) ring (C_6H_6); it is a glassy, brittle polymer with low toughness and relatively low softening temperature and because of its easy processing its uses are widespread as a commodity plastic, for example in packaging applications. In cellular form, it is an important insulation material. Poly(methyl methacrylate) (PMMA) is another polymer with a simple carbon backbone containing in its monomer a methyl CH_3 and a methacrylate group $COO–CH_3$; it is a glassy, transparent, tough material with high resistance to outdoor weathering; it is used extensively as a replacement for glass in constructions. It is important to note that if the molecular structure is characterised by a high order by placing the substituent groups or atoms at the same place in all monomer units, micro/mesoscale structures of high order such as crystalline domains are promoted. This has important consequences for various physical properties which are important in applications. To give an example, if the methyl substituent in PP is attached in an ordered fashion, isotactic PP is produced with high degree of crystallinity. This is the most useful form of PP. On the other hand, PS is used in its disordered atactic configuration and it is amorphous.

Polyoxymethylene (POM) and related polymers incorporate oxygen within the polymer chain backbone; they have high strength and stiffness. Polycarbonate (PC) has a more complicated molecular structure containing aromatic rings along the backbone; it is also very tough glassy polymer with good mechanical properties at a range of temperatures, good dimensional stability, and resistance to burning, environmental and chemical conditions. It is used widely in transparent roof panels. Poly(ethylene terephthalate) (PETP) and poly(butylene terephthalate) (PBTP) include the ester link –O–CO– in the backbone; they have good insulating and permeation properties and fire resistance, and are used in the form of thin sheets for insulation and sealing in buildings. Polyamides (PA) (nylons) contain the amide link –NH–CO– within the backbone; they are widely used polymers in various engineering applications from bearings and gears to ropes and pipes.

Polymers such as polybutadiene, polyisoprene and polychloroprene are liquids with high viscosity at room temperature (the viscosity increases with the degree of polymerisation) and they become solids characterised by rubbery behaviour if the polymer chains are lightly cross-linked (cured), that is, connected covalently by chemical reaction (adding sulphur which is called vulcanisation) or by other means (UV radiation, electron beam processing or simply by heating). If the number of cross-links is high, a thermoset material forms, characterised by relatively high stiffness and strength in addition to toughness. Thermosets are usually formed by cross-linking short chains or even directly the monomer units which contain at least three binding sites. Epoxy resins (EP) and phenol-formaldehyde resins (PF) are thermosets, and are used in many structural applications reinforced by mainly glass fibres. Formaldehyde resins are used extensively as wood adhesives.

Copolymers incorporate two or more different types of monomers within the same macromolecule. In this way, one can create new materials with the desired combination of properties. Styrene-butadiene-rubber (SBR) combines the (flexible and mobile) liquid-like character of polybutadiene and the (rigid) glassy behaviour of styrene at room temperature to produce physical cross-links between the

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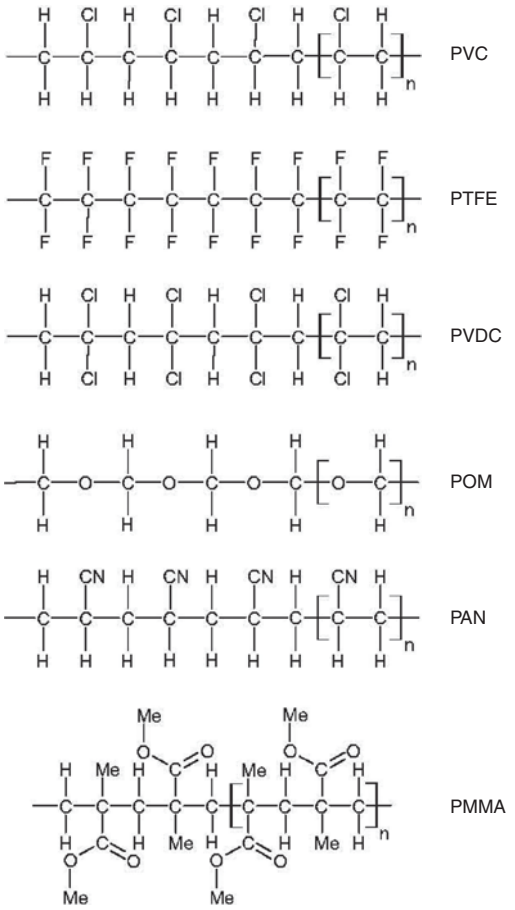

Figure 9 Some halopolymers and heterochains

copolymer chains by segregation of the two types of monomers. In this way, we produce a material which is rubber at room temperature but it can melt at high temperature and solidify at room temperature reversibly and repeatedly. Thus, unvulcanised SBR is a thermoplastic elastomer (TPE).

Ethylene monomers can be combined with propylene monomer units to produce poly(ethylene-propylene). Depending on the composition, the resulting copolymer can behave from modified polyethylene to modified polypropylene. A simpler and inexpensive way to tailor the desired properties is to physically blend two or more types of polymers. The resulting microstructure and consequently the physical properties depend crucially on the ability of the polymers to blend homogeneously at the molecular scale. High-impact polystyrene (HIPS) consists of a PS matrix and dispersed polybutadiene rubber particles (**Figure 4**), since PS and polybutadiene are immiscible. PPO-PS and PETP-PBTP are examples of miscible blends.

Polyacrylamide is a water-soluble polymer. At a solution of only 3% in water it can be chemically cross-linked and form a polymer gel which has some intriguing properties. It can behave like a viscous fluid during shearing/pumping and like a soft solid at rest. It has some uses as a chemical grout for soils.

The polymer solid state

Above a certain temperature all polymers soften and, in the case of thermoplastics, are able to flow (they become a melt). At this raised temperature (for common plastics usually in the range of 100–250°C) the polymers have liquid-like order, i.e. they are disordered and in the melt state. As the temperature drops, their density increases, i.e. specific volume decreases (**Figure 10**). Depending on the polymer, there are two possibilities: (1) polymers with irregular molecular structure (atactic PS, atactic PP, PMMA) solidify keeping their disordered microstructure and forming a stiff but brittle amorphous solid called polymer glass; the specific volume-temperature slope (**Figure 10**) changes in a continuous fashion at the specific temperature of the transition and for this reason it is called glass transition temperature, T_g; (2) polymers which have a regular structure at the molecular scale (PE, PEO, isotactic PP, isotactic PS, PA, PTFE, PETP) crystallise (partially) forming a semi-crystalline material in an abrupt manner (**Figure 10**) at a characteristic temperature called crystallisation temperature, T_c. For polymers (unlike crystals of small molecules), the crystallisation temperature might differ from the melting temperature, T_m, by several degrees. Usually $T_c < T_m$, an effect called undercooling. Furthermore, both temperatures depend on the rate of cooling/heating. This is because long polymer chains, unlike small molecules, have decreased flexibility and are easily trapped in kinetically arrested states. It is difficult (or even impossible) for them to attain their absolute thermodynamic equilibrium state of full crystallinity. High degrees of crystallinity can be attained in the case of very slow cooling rates. If they are cooled abruptly enough, they freeze in the fully disordered state forming a polymer glass.

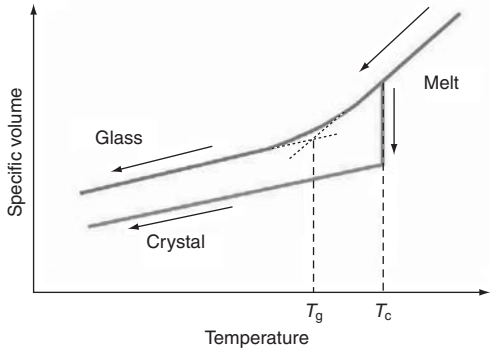

Figure 10 Cooling down from the melt state to glass or to crystal. Note the relatively smooth transition to glass compared to the abrupt crystallisation

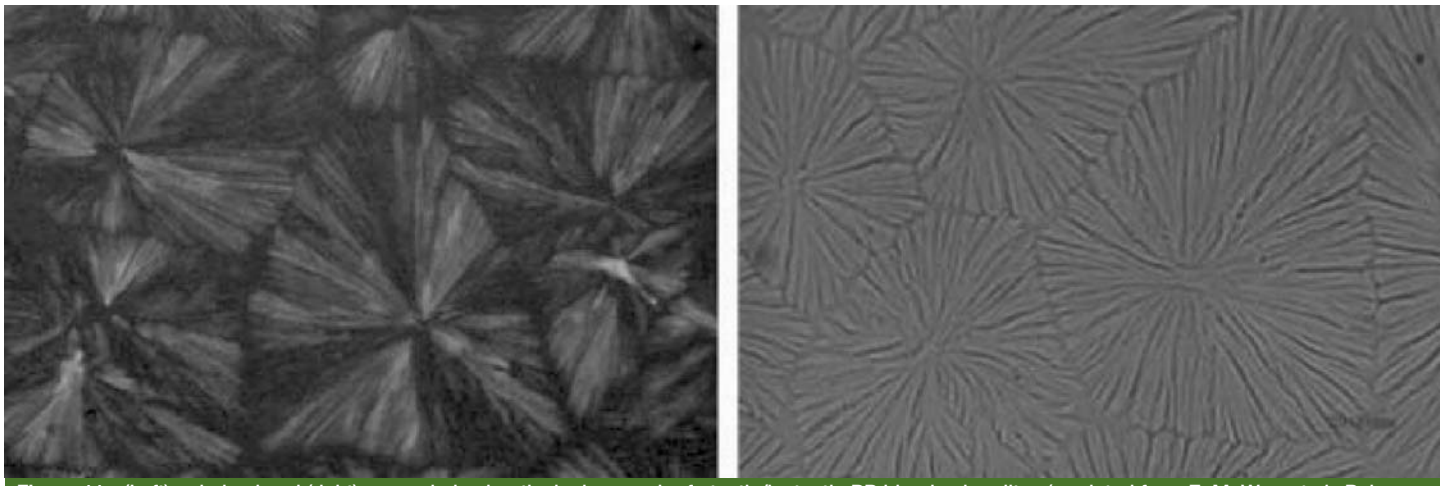

Figure 11 (Left) polarised and (right) non-polarised optical micrograph of atactic/isotactic PP blend spherulites (reprinted from E. M. Woo *et al*., Polymer ©2007 Elsevier)

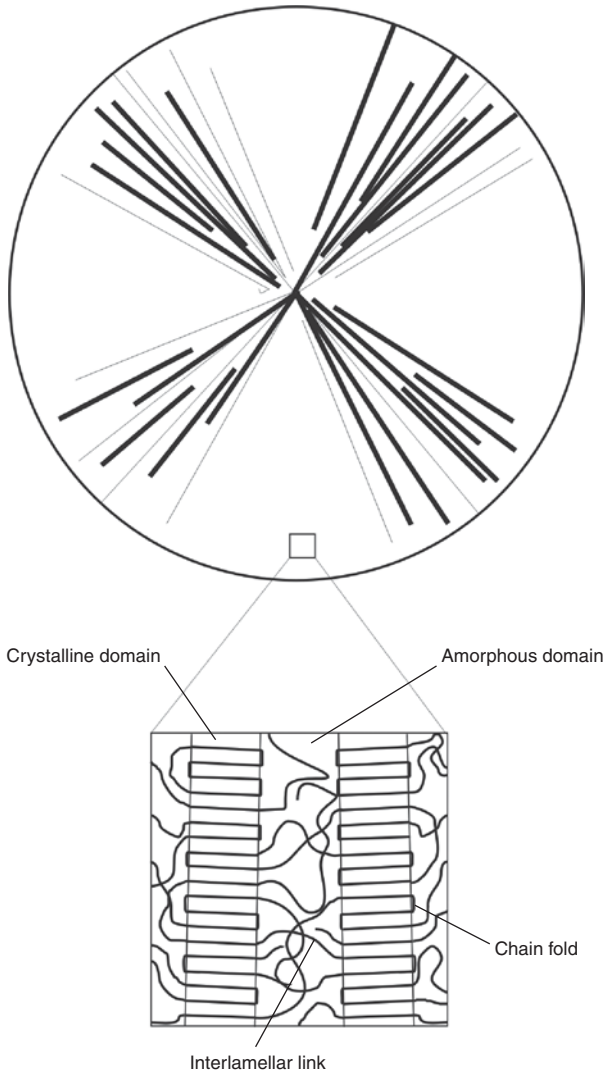

Figure 12 Spherulite organisation

A typical crystallinity of LDPE is in the order of 50%, while HDPE can be up to 90% crystalline. The crystal 'grains' (crystallites) in polymers take a roughly spherical shape (**Figure 11**) and they are called spherulites. However it has to be noted that, unlike grains in metals, spherulites contain both crystalline and amorphous domains organised in a radial lamellar structures (**Figure 12**).

The crystallites form and grow within the melt state during cooling down and their morphology depends strongly on processing history such as the rate of cooling. The morphology, and consequently the processing, affects significantly the physical properties of the final material. Within spherulites, a typical polymer chain belongs to both crystalline and amorphous domains connecting efficiently the whole structure. Furthermore, in partially crystalline polymers, the crystallites play the role of physical cross-links within an amorphous matrix. If the temperature of use is below T_m and above T_g, the material is tough with a leather-like character.

The crystalline state is characterised by higher densities, and improved mechanical properties. Semi-crystalline polymers capable of a high degree of crystallinity (PE, PP, PA) are used for the production of fibres which contain highly oriented crystalline domains in the direction of the fibre axis by cold drawing. The axial elastic modulus and strength of these fibres are significantly improved since the stress is opposed by the strong primary bonds of the macromolecular backbone. The mechanical properties of PVC pipes are also enhanced in a similar way. After the extrusion process the pipes are radially expanded in order to enhance the orientation of polymer chains in the hoop direction.

Thermoset polymers and gels lack organisation at all scales and are inherently amorphous. However, elastomers are capable of high strain behaviour which induces chain orientation and alignment; a process which can result in

temporary crystal formation with consequences on their physical properties. Thus many elastomers become stiffer at high extensions.

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Chapter 47

Polymer engineering

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Distinctive processing technologies have been developed for polymers using low temperature moulding, extrusion and casting methods. These allow components of complex and precise shape to be formed.

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Introduction

Polymer materials are (with few exceptions) carbon-based materials, produced largely from gas and oil raw materials. There are routes to polymers from wood and other natural materials, but apart from natural rubber, these do not contribute significantly to consumption in the polymer sector.

Polymerisation reactions

Solid thermoplastics are produced by the primary materials supply industry in large-scale chemical plant. The chemical reactions are specialist matters for chemical engineers and polymer chemists. The syntheses of the major commodity polymers such as the polyolefins polyethylene and polypropylene (PE and PP) make use of hydrocarbon feedstocks such as naphtha or natural gas, reformed to provide the monomers used in polymerisation. The process routes to these polymers usually employ advanced technology, for example the use of Ziegler-Natta catalysts containing titanium metal compounds. Catalysts are used to tailor the characteristics of the polymer chain such as stereoregularity, chain length and its distribution and branching, in order to achieve particular engineering properties. The products of the primary polymer synthesis are then passed in the form of granules and powders to the thermoplastics processing sector for conversion to a wide variety of end-user forms and products.

In contrast, the formation of thermoset polymers occurs much closer to the point of application, perhaps on site or in the fabrication shop. Thermoset polymerisations occur in forming reinforced plastics such as fibre-reinforced epoxy or polyester components, and in the setting of two-pack adhesives or coatings.

Broadly speaking, the chemical composition of the thermoplastics is simpler than that of the thermosets. We have described the primary structure of several thermoplastic polymers in the previous chapter. These are linear molecular chains of repeating units. On the other hand, the thermosets by definition are three-dimensional random networks. Some are formed in a single step by reacting together two (or occasionally more) reactive monomers to form a network. Such is the case for the phenol-formaldehyde resins. In other cases a precursor polymer chain (usually of rather short length) is formed and then subsequently cross-linked with a second component. Occasionally, the cross-linking agent may be moisture or oxygen from the atmosphere (as in cyanoacrylate adhesives or some polyurethanes). These reactions may be initiated or controlled by additional components (*activators*, *initiators* or *curing agents*, for example), so that the formulations may be complicated and multicomponent. In any case, since the mixing of the components may take place on site, there are quality control challenges in using such materials. The monomers are generally reactive and frequently volatile, so that health and safety considerations also arise.

Epoxies

The epoxy resins are an extremely important and versatile family of thermoset polymers. In **Figure 1** we show schematically how these resins form the cross-linked network present in epoxy structural adhesives, composites and coatings. The two main components are a diphenol (usually bisphenol A) and an epoxide (commonly epichlorhydrin). The phenol molecules are linked together by bridging epoxide molecules to form a short chain (with up to a dozen phenol units). These short precursor polymer chains have unreacted epoxide groups at each end, and can then be formed into networks by a further reaction between the terminal epoxide groups and a cross-linking agent such as an amine. This rather complicated process can be varied and modified in innumerable ways, for example by varying the structure or chain length of the epoxy prepolymer or the structure of the cross-linking agent, of which there are several hundreds; or the

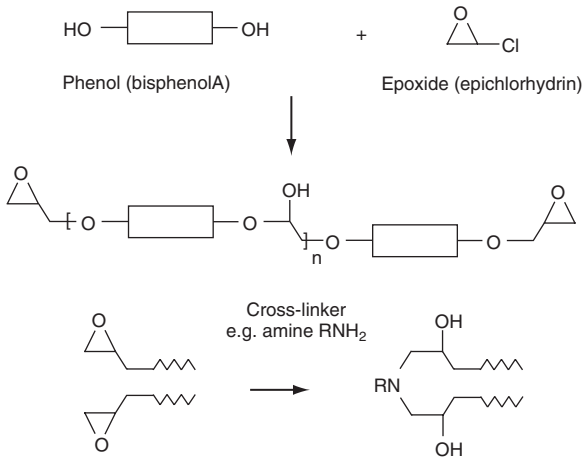

Figure 1 Schematic of epoxy curing reaction

concentration of cross-linker. Furthermore, the formulated epoxy resin may contain other components, notably fillers and flame retardants. The epoxies have excellent resistance to chemical attack, good adhesion to many substrates and excellent mechanical properties (strength, elastic modulus, toughness). The epoxy curing reaction, unlike that of polyesters, does not produce water or other volatile by-product, thus largely eliminating shrinkage during hardening.

Unsaturated polyesters

Polyesters are a large and diverse family of heterochain polymers (the primary chain contains oxygen atoms as well as carbon atoms). The unsaturated polyesters are an important sub-group. Unsaturation in the main chain is provided by C=C double bonds and this allows cross-linking of the linear chains by a hardening agent such as styrene. The unsaturated polyesters are thus thermosets and are the base polymers for many fibre-reinforced plastic products and components.

Compounding of polymers

Polymer materials are rarely used in pure form, but usually are supplied in a variety of types and grades in which the base polymer is combined or *compounded* with a number of property-modifying additives. Commonly, base polymer is mixed with *fillers* such as talc, silica, glass powder or fibre, or graphite. Fillers may serve just to bulk the material and reduce unit cost, or provide colour or opacity, but can also serve to increase stiffness, hardness and durability, or to reduce thermal expansion. In addition, many other functional additives may be incorporated in small quantities: for example, flame retardants to modify fire performance, antimicrobials to prevent biological attack, and plasticisers to increase flexibility. Some additives, such as lubricants and antiblock agents, modify process properties during extrusion or moulding.

Processing methods

The main methods of shaping and forming thermoplastics are by extrusion and moulding. Several distinct moulding processes are used, including injection and rotational moulding, vacuum forming and thermoforming. Most thermoplastics have low melting temperatures (100–200°C) and the polymer is generally processed in the molten state at temperatures 30–50°C above the melting point. The melt viscosity (which may be high) is an important factor in processability.

The extrusion of molten thermoplastics such as PE, PP through a shaped die provides the means of producing any polymer product with a constant cross-section, such as tube, rod or geometrically complicated profiles, for example for ducting. Sheets and films are generally produced by extrusion, and extrusion is the standard method of wire coating (cable insulation). The viscoelastic behaviour of thermoplastics leads to significant expansion as the polymer emerges from the die and cools. Such *die swell* is allowed for in the tool design. A variant of extrusion in which continuous fibre reinforcement is co-extruded with the polymer melt is known as *pultrusion*.

Calendering is the term used to describe the continuous production of sheet by passing polymer feed between rollers. It is well suited to form multilayer or textile reinforced sheets, or sheet with a textured finish.

In *injection moulding*, polymer melt is forced under pressure into a closed mould, cooled and ejected. The use of pressure injection allows rapid cycle times, although the low thermal conductivity of polymers may limit cooling rates. Intricate products can be formed with excellent finish. Injection moulding is generally employed for small components. Related processes include *blow moulding*, in which a section of extruded tube is inflated in a two-part mould to produce bottles, containers and small tanks, and *rotational moulding*, also for containers and tanks, in which polymer powder is melted and distributed on the inside of a heated mould by tumbling.

Vacuum forming is a widely used method of shaping components and products from polymer sheet. The sheet is softened rather than melted, and then drawn into contact with a mould by using a vacuum. In *thermoforming*, air pressure or a shaped plug is used to force the softened sheet into contact with the mould. There are many variants of these forming processes.

Composites

Thermoset polymers are combined with glass fibre, carbon fibre and other reinforcements to produce com-

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posite materials by a variety of moulding processes. The methods of making polymer composites are fully described elsewhere.

Cellular polymers

Most solid polymers can be produced as foams, generally in the form of sheet materials. These foamed or cellular polymers have exceptionally low densities and low thermal conductivities, and so find use as insulation materials. It may also be that a foamed material has greater stiffness than the same amount of the same material in a thin sheet. Foams are therefore often used in sandwich panel structures to provide rigidity with little weight penalty. Moulded granules can also be produced as loose fill. Both flexible and rigid cellular materials are manufactured. The polymers used most extensively in cellular form are thermoplastics such as polyolefins (PE and PP), and polystyrene PS; thermosets, notably the phenolics, and urea-formaldehyde (UF); and elastomers such as natural rubber and various synthetic rubbers. Within the diverse polyurethane PUR group there are both rigid and flexible (elastomeric) foams.

There are broadly three routes to cellular polymers: (1) by mechanical agitation of a polymer emulsion or partly polymerised liquid resin; (2) by use of a physical blowing agent, such as a halocarbon gas; and (3) by means of a chemical blowing agent which decomposes to yield a gas. It is useful to distinguish between closed cell and open cell foams (**Figure 2**). Method (1) produces open cell foams (essentially sponges), which have high permeability to liquids and gases. Closed cell foams are produced by methods (2) and (3). They cannot absorb water into the pores, have excellent barrier properties and long-term buoyancy. Equally significant is that their mechanical properties differ sharply from those of open cell foams because air is trapped within the pores and resists compression.

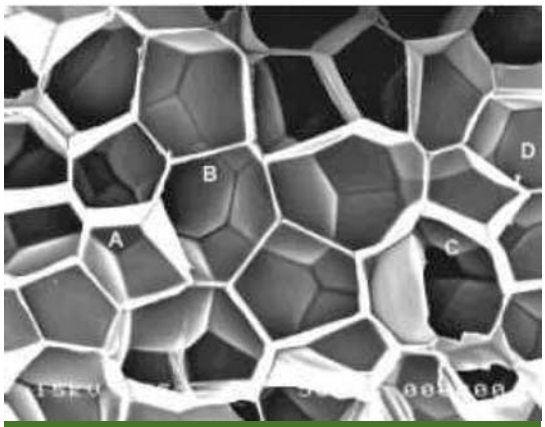

Figure 2 The structure of a polymer foam (reprinted from O. Almanza *et al*., Polymer Science ©2004 John Wiley and Sons)

Sheet, mesh and grid

The production of continuous sheet by calendering has already been described. However, most sheet is produced by extrusion through a slot (flat sheet) die. Both methods can produce multilayer and textile reinforced materials.

It is in the production of grids, meshes and non-woven textiles that there has been interesting innovation. An important step was the development of the Netlon and Tensar processes for meshes and grids. In the Netlon process extruded polyolefin sheet is slit with rotating cutters as it emerges from the die and is then subjected to biaxial stretching to form a mesh and to stiffen the ligaments by chain orientation. In the later Tensar process (**Figure 3**), a similar concept is applied to process thicker sheet (say 5 mm) which is punched to form a pattern of circular holes before strong biaxial stretching. This forms stiff, strong grids of the kind now widely used in soil reinforcement. By such biaxial orientation it is possible to achieve tensile strengths as high as 500 MPa (Carter and Dixon, 1995).

Emulsions and gels

With few exceptions, engineering polymers are insoluble in water. In use this is beneficial, but it means polymer solutions required as the main component of coatings and adhesives must employ organic solvents. An important way to make a water-based liquid form of polymer is to disperse the polymer to form an aqueous emulsion. This is not a true solution but a mixture of polymer particles in water. Natural rubber latex as harvested from the rubber tree is such an emulsion. Most commercial polymer emulsions use either acrylic or polyvinyl acetate (PVAC) or styrene-butadiene polymers. The chemistry is complex and copolymers are generally used. Particles are typically 1–5 μm diameter (**Figure 4**). They are stabilised chemically by the addition of surfactants to prevent aggregation. A polymer emulsion (or *latex*) may contain 50–60 wt per cent of polymer solids. Such emulsions form the main component of many surface coatings and paints, and of commodity adhesives.

Coatings and paints

Paints are among the most compositionally complicated polymer-based materials. Paint formulations may contain ten or more components, each contributing to the performance of the coating at every stage from application as a wet film, during film formation and throughout its service life. Normally there is a base polymer, solvent or dispersing medium, pigment (or several), dispersing agent and other additives to ensure the stability of the dispersion in transport and storage, additives to improve

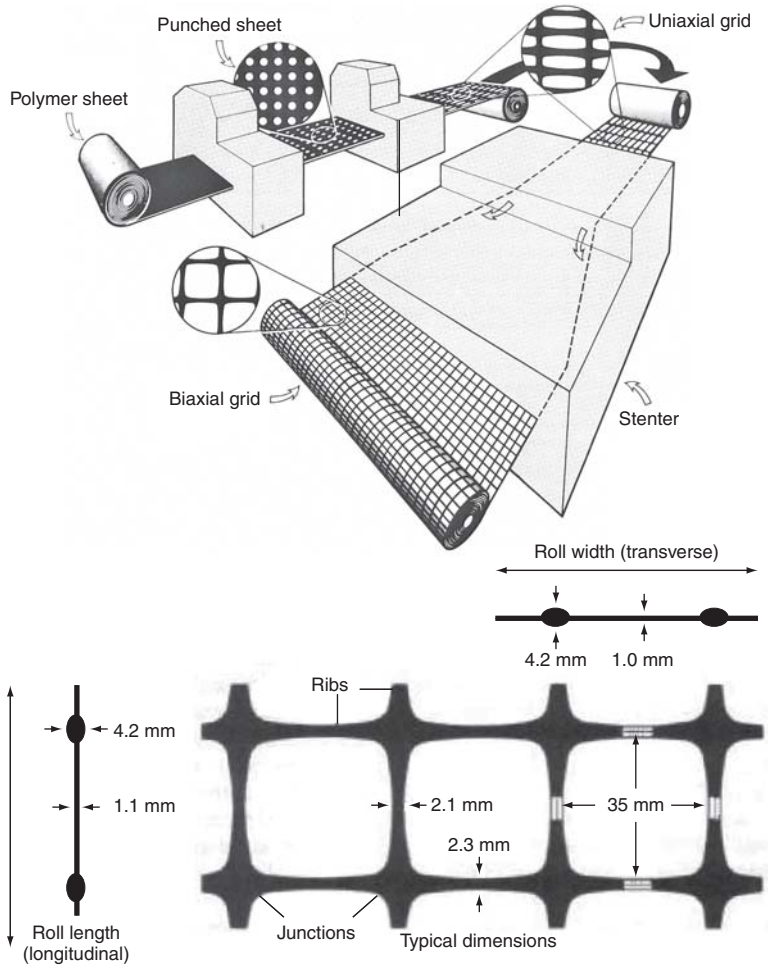

Figure 3 The Tensar process for forming polymer grids (courtesy of Tensar International Ltd)

spreading and to control viscosity during application. There may be an additive also to provide some *thixotropy* to the paint to minimise sag after application and before drying. Paints based on emulsions may contain a volatile plasticiser to help particles coalesce into a continuous film during drying. Finally, the paint may contain anti-oxidants to reduce degradation when exposed to oxygen and light.

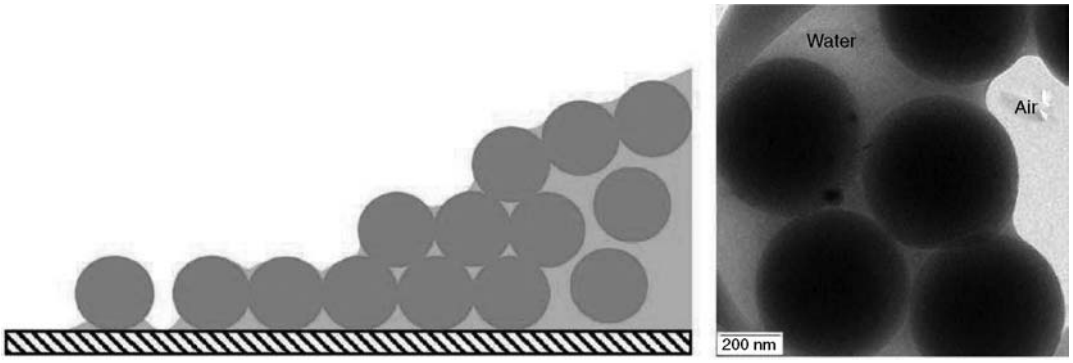

Figure 4 Spherical polymer particles in a water emulsion (reprinted from Y. Ma, H. T. Daris and L. E. Scriven, Progress in Organic Coatings ©2005 Elsevier)

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Chapter 48

Engineering properties of polymers

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Polymer materials are generally softer and weaker than metals and ceramics, but have distinctive engineering properties of great practical value.

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Introduction

The most important properties for polymeric materials used in civil engineering are undoubtedly those to do with their response in stresses and strains – that is, the properties that determine mechanical behaviour. In this chapter, we give a concise account of the mechanical properties of polymeric materials related to standard mechanical testing such as uniaxial tensile testing, creep and stress relaxation, and more complicated processes such as friction, wear and impact. The mechanical behaviour of polymeric materials is fundamentally linked to thermal behaviour, so a brief account of thermal properties is also provided. Furthermore, we discuss some other properties which are of interest to a civil engineer such as permeability and durability, which usually depends on environmental conditions.

Mechanical properties

Elastic, viscous and viscoelastic response

The strain response of a material over the passage of time due to the application of a constant load is called creep (**Figure 1(a)**). A purely elastic material responds instantaneously to the load and the strain remains constant; furthermore, it will recover its initial shape instantaneously upon the removal of the load (**Figure 1(b)**). On the contrary, a viscous liquid will deform as long as the load continues to be applied. Upon removal of the load, the fluid does not return to its initial position (**Figure 1(c)**). Fluids show a characteristic resistance to movement (flow), which is called viscosity. Viscosity results in a frictional energy loss, which dissipates in the fluid as heat. Polymeric materials behave both as viscous fluids and elastic solids. They are viscoelastic materials. The most important characteristic of viscoelastic materials is that their mechanical properties depend on time. The response of a viscoelastic material is intermediate between the solid and the liquid (**Figure 1(d)**). There is usually an instantaneous elastic response followed by a delayed elastic response that could be followed by a purely viscous response. The creep recovery which follows upon the removal of the load starts with the immediate recovery of the instantaneous elastic response followed by the slow and gradual recovery of the delayed one; the viscous part does not recover. Creep and recovery depend on the applied load, molecular characteristics, microstructure and temperature.

Uniaxial tensile testing

As we have already indicated, the mechanical behaviour of polymeric materials depends strongly on temperature.

Amorphous thermoplastics are stiff and strong at temperatures below the glass transition temperature. For many polymers used in civil engineering applications such as poly(methyl methacrylate) (PMMA) and polycarbonate (PC) the glass transition temperature is in the range 100–200°C which means that there is a useful temperature range near room temperature where amorphous thermoplastics are glassy with high Young's modulus and strength (see **Figure 2(a)**). However, one has to note that they are brittle, failing by catastrophic crack propagation at relatively (for polymers) low strains (2–5%). At higher temperatures as the temperature approaches glass transition, the materials starts to soften, creep and behave in a viscoelastic manner. Ultimately, above glass transition the material becomes a viscous liquid.

Thermosets, which are highly cross-linked networks, exhibit similar mechanical behaviour to glassy thermoplastics. At higher temperatures they also soften but never reach the state of viscous flow due to the cross-links which sustain cohesiveness.

Semi-crystalline polymers at room temperature are usually above glass transition and below the melting temperature, and for these reasons they are less stiff but tougher. Their Young's modulus is not as high as glassy polymers and thermosets and they creep considerably if not reinforced. Viscoelastic behaviour is quite usual at room temperature for semi-crystalline polymers and their stress–strain behaviour is dominated by yielding which leads to necking and in many cases strain hardening at higher strains due to the orientation of the crystalline

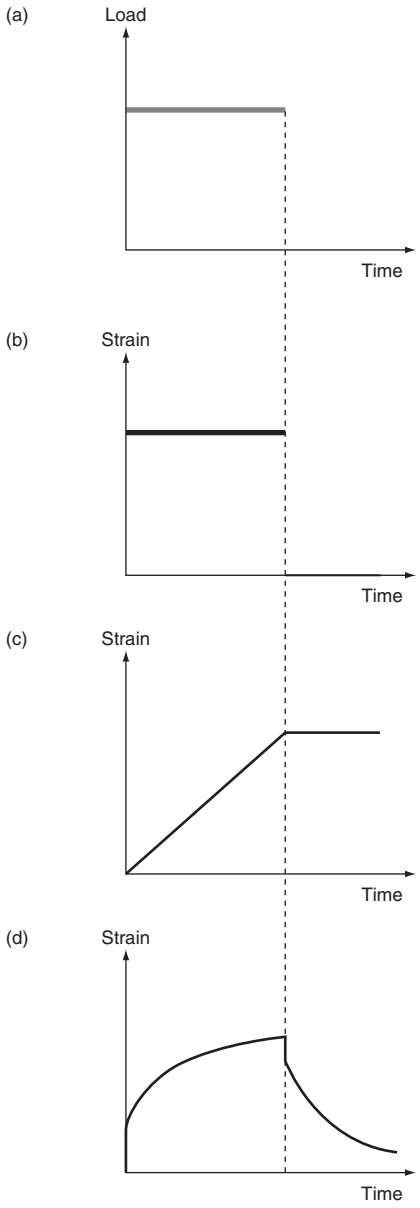

Figure 1 Strain response of a material subjected to a constant load for a finite time interval (up to the dashed line). (a) Load application–creep test; (b) solid: elastic behaviour; (c) liquid: viscous flow behaviour; (d) polymer: viscoelastic behaviour

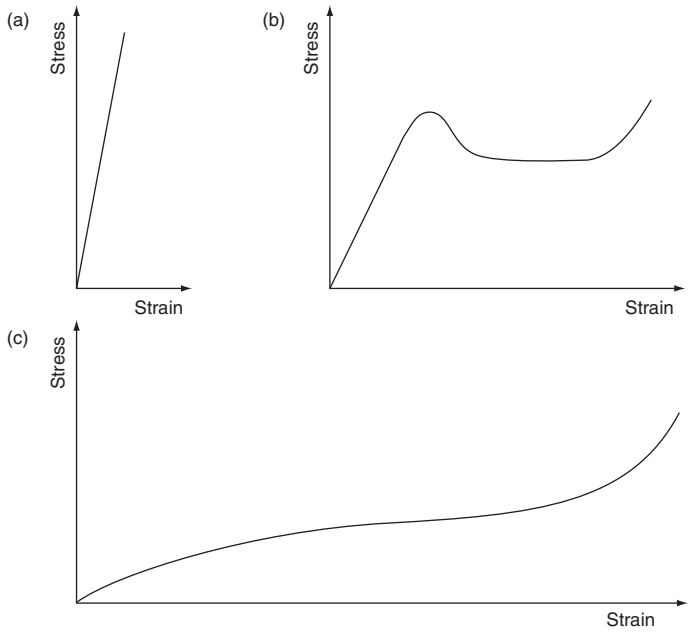

Figure 2 Typical stress–strain curves (uniaxial tensile testing) for (a) a polymer below T_g; (b) a semi-crystalline polymer above T_g; (c) a rubber

domains (**Figure 2(b)**). As temperature increases the material softens and ultimately above the melting transition temperature it becomes a viscous liquid. Conversely, if the temperature falls below the glass transition temperature, the modulus increases and the material becomes brittle.

Elastomers above the glass transition (which is usually below 0°C) are characterised by low elastic modulus (**Figure 2(c)**) but their main characteristic is the extremely high strains which can be attained. Recoverable deformations of 1000% are not unusual before strain hardening and failure. Although they never become completely viscous, in the vicinity and above glass transition, elastomers exhibit time-dependent behaviour and so-called *retarded elasticity*, i.e. viscoelasticity. At sufficiently low temperatures, they become glassy, i.e. stiff and brittle.

As we can see, a general characteristic of polymeric mechanical behaviour, which in many cases is pronounced at room and higher temperatures, is the time/frequency dependence of the mechanical properties. This is manifested in various mechanical tests, e.g. different stress–strain curves in tensile testing when either the strain rate or the temperature changes; intermediate behaviour between a viscous liquid and an elastic solid in creep and stress relaxation tests; or phase lag and hysteresis in dynamic mechanical testing.

Generally, polymeric materials exhibit a Young's modulus and strength of approximately two orders and one order of magnitude respectively lower than that of metals. However, one has to note that in general their densities are almost an order of magnitude lower, which makes them ideal for applications when the ratio of mechanical properties to weight is of paramount importance. Some mechanical properties of selected polymeric materials are given in **Table 1**.

Creep, recovery and stress relaxation

A direct manifestation of time-dependent behaviour in polymeric materials is their tendency for creep at room temperature; this is particularly pronounced for thermoplastics. In **Figure 3(a)** we show schematically a family of typical creep curves: strain–time curves at constant stress.

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Polymer	ρ: g/cm^3	E: GPa	σ_F: MPa	ε_F: %	σ_Y: MPa	ε_Y: %	T_g: °C	T_m: °C
PS	1.05	3.1–3.3	30–55	1.5–3	–	–	90–100	–
PC	1.20–1.24	2.2–2.4	55–65	100–130	55–65	6–7	145	–
PMMA	1.15–1.19	3.1–3.3	60–80	2–6	–	–	105–120	–
PVC	1.38–1.55	2.7–3.0	50–60	10–50	50–60	4–6	80	–
PP	0.90–0.91	1.3–1.8	25–40	>50	25–40	8–18	0–20	160–165
HDPE	0.94–0.96	0.6–1.4	18–30	>50	18–30	8–12	⩽−100	125–135
LDPE	0.91–0.93	0.2–0.4	8–10	>50	8–10	20	⩽−100	100–110
PA 6	1.12–1.15	2.8	80	30	80	4	78	230
EP	1.17–1.25	<4.2	<100	1.5–20	–	–	70–200	–
UP	1.2	3.2–3.5	50–77	1.2–2.5	–	–	70–150	–

Table 1 Mechanical properties of some polymeric materials (adapted from Ehrenstein, 2001)

In stress relaxation where the stress is monitored at constant strain, the stress diminishes progressively with time. Creep test data are used extensively in determining the mechanical behaviour of polymeric materials for design. A major reason is that technically it is easier to implement the condition of constant stress (for example at constant tensile load) than the condition of constant strain needed for stress relaxation experiments. The strain–time relationship is approximately a power law which translates to an approximate linear relationship on a double logarithmic plot.

Creep curves (see **Figure 3**) can be transformed easily to isochronous stress–strain curves and isometric stress–time curves. It has to be noted that isochronous stress–strain curves are generally different from tensile testing stress–strain curves and similarly, isometric stress–time curves are generally different from stress relaxation-time curves because the testing conditions and consequently the mechanical history are different. However, due to the simplicity of creep tests, creep data are often used in design as adequate approximations. In many cases, at sufficiently low strains, the stress is proportional to strain which means that the creep compliance (or creep modulus) provides the full mechanical response of the material at any stress by using Hooke's law, $D(t) = 1/E(t) = \varepsilon(t)/\sigma$, and we say that the material is linear viscoelastic.

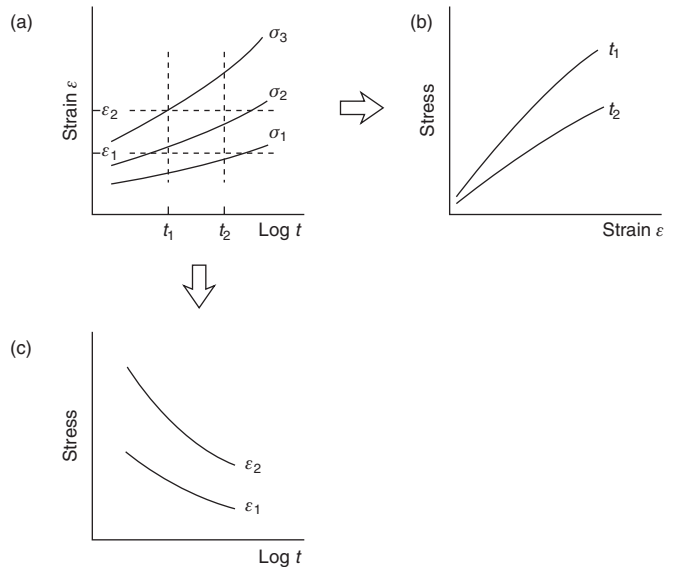

Figure 3 A family of (a) creep (strain–time) curves; (b) isochronous stress–strain curves; (c) isometric stress–time curves

After a creep test, upon the removal of the load (or generally constant stress), the strain tends to slowly approach its original value although it is not usual for polymers to recover fully.

At high stresses the creep response of a polymeric material becomes non-linear and the creep compliance or modulus becomes a function of strain (**Figure 4**). Ultimately, after a period of creep (which can be very long) the polymer fails. Polymers can fail in various ways. Brittle fracture is usual for stiff/rigid and strong thermoplastics and thermosets while ductile yielding is the mechanical failure mode for semi-crystalline polymers. It has to be stressed again, though, that the behaviour can vary from these two extremes with temperature and strain rate in a dramatic fashion. Consequently, the mechanical properties are a function of temperature and time/frequency of deformation. We note that extrapolations at long times using a power law (that is, a linear relationship in double logarithmic plot), between stress and creep strain is not always advisable since for some polymers the rate of creep accelerates further at long times.

Elements of linear viscoelasticity theory and empirical relationships

We can use combinations of springs (linear elastic behaviour) and dashpots (linear viscous behaviour) in

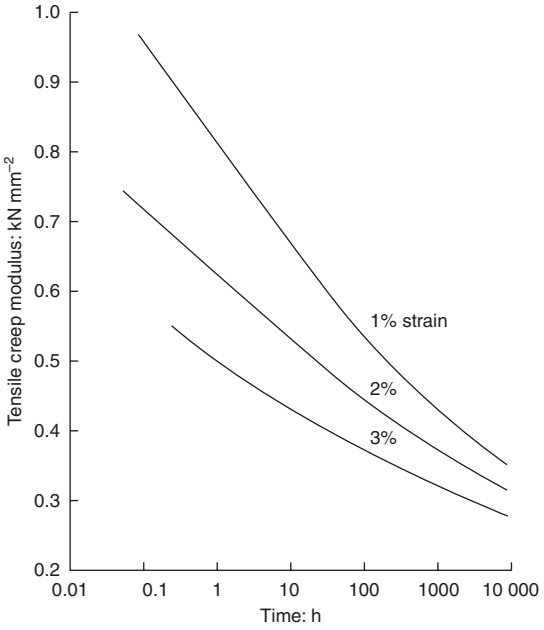

Figure 4 Strain dependence of creep modulus

order to quantify the mechanical behaviour of polymeric materials. In **Figure 5** the *Voigt* (or *Kelvin*) *model* is shown. This is a parallel combination of a spring and a dashpot. The total stress σ is distributed both to the spring σ_1 and to the dashpot σ_2: $\sigma = \sigma_1 + \sigma_2$ and the strain is the same for both spring and dashpot: $\varepsilon = \varepsilon_1 = \varepsilon_2$. The linear elastic behaviour represented by the spring is given by $\sigma_1 = E\varepsilon_1$, where E is the elastic modulus. For the viscous liquid behaviour represented by the dashpot we have $\sigma_2 = \eta\, d\varepsilon_2/dt$, where η is the viscosity. Thus, we obtain the following differential equation: $\sigma = E\varepsilon + \eta\, d\varepsilon/dt$. Solving this for constant stress (creep test, $\sigma = \sigma_0$), we obtain the creep solution: $\varepsilon(t) = (\sigma_0/E)[1 - \exp(-t/\tau_t)]$ where $\tau_t = \eta/E$ is the characteristic relaxation time. If at $t = 0$, $\varepsilon = 0$, then as $t \to \infty$, $\sigma \to \sigma_0/E$ asymptotically (**Figure 6**). This model can represent creep but without the instantaneous initial

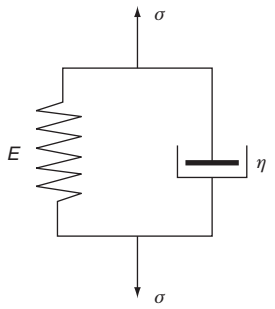

Figure 5 Voigt (or Kelvin) model of linear viscoelasticity: combination of a spring and a dashpot in parallel

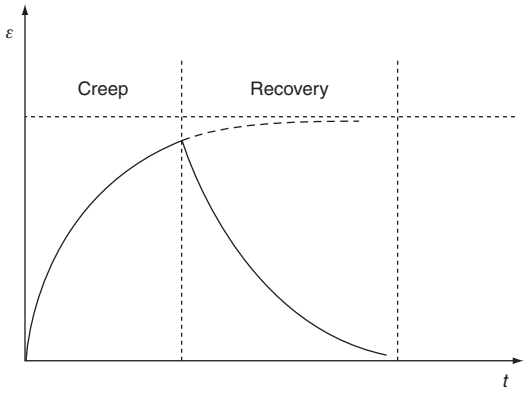

Figure 6 Creep and recovery for the Voigt (or Kelvin) model

deformation. For recovery (removing the stress, $\sigma = 0$) at a new $t = 0$, $\varepsilon = \varepsilon_1$ and $\eta\, d\varepsilon/dt = -E\varepsilon$. This has a solution $\varepsilon = \varepsilon_1 \exp(-t/\tau_t)$. The strain goes asymptotically to 0 with the same relaxation time. The recovery does not include the instantaneous deformation and a percentage of permanent deformation. This model cannot account at all for relaxation (for $\varepsilon = \varepsilon_0$, $\sigma = E\varepsilon_0 =$ constant, it does not relax). The relaxation can be modelled by the Maxwell model where the spring and dashpot are positioned in series. In this case, the relaxation stress decays as a single exponential $\sigma = \sigma_0 \exp(-t/\tau_t)$. A more realistic model is the *standard linear solid* which consists of a linear spring in series with a Voigt model. Furthermore, generalised linear viscoelastic models which consist of combinations of a spring, a dashpot and many Maxwell or Voigt models can be constructed for the analysis of polymeric materials with a spectrum of relaxation times.

In addition to viscoelastic models, many empirical equations have been proposed for creep. One which applies to some of the common engineering plastics has the form $\varepsilon(t) = K\sigma t^n$, where n and K are constants for a given polymer and $0 \leqslant n \leqslant 1$. In cases where $n = 0$, the material behaves in a purely elastic manner. Alternatively, at $n = 1$ the material behaves as viscous fluid. The value of n obtained from creep data is therefore a measure of the relative contributions of elastic and viscous deformation to the creep process.

Molecular scale origins of mechanical behaviour

The time-dependent behaviour (creep, recovery, stress relaxation) is a direct consequence of the macromolecular character of the polymer molecules and the weak physical interactions between them (weak attractive van der Waals forces). The long polymer chains move/relax at slower rates compared to simple liquids. The mobility of long chains is due to a relatively slow serpent-like movement called reptation. The mechanical response of a material

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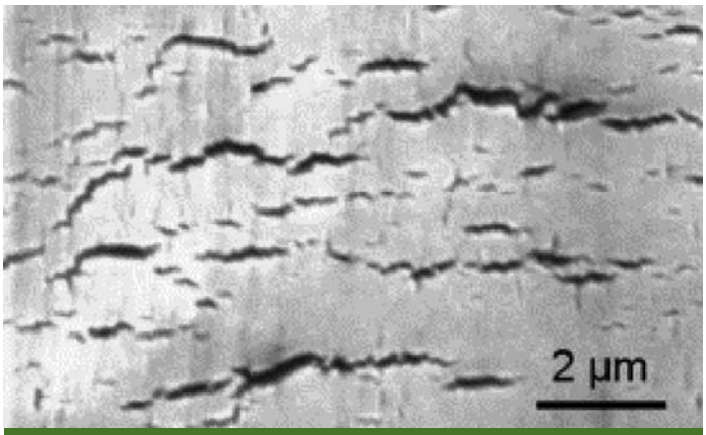

Figure 7 Crazes in a poly(ethylene terephthalate) PET sample under uniaxial tensile tension. The crazes are perpendicular to the tensile stress (reprinted from C. G. Sell *et al*., International Journal of Solids and Structures ©2002 Elsevier)

depends on the time it takes for its individual molecules to respond in the imposed deformation/stress, i.e. the molecular *relaxation time*. If the rate of deformation is slow compared with this characteristic time and allows enough time for the molecules to move and relax, the response is liquid-like. In contrast, if the rate of deformation is high compared with this characteristic time and does not allow enough time for the molecules to move and relax, the response is solid-like. Liquid-like behaviour favours flow, retardation and consequently yielding while solid-like behaviour favours elastic response and ultimately crack propagation and brittle fracture. Intermediate types of behaviour are possible with different polymers and also with the same polymer in different temperatures and deformation rates. Furthermore, the crack formation and propagation in polymers is usually preceded by the formation of *crazes*, regions of *fibrils* and voids which dissipate energy efficiently even though macroscopically the failure is brittle (**Figure 7**). Crazing is a process which occurs only in polymer materials and is a direct consequence of their macromolecular nature.

Impact testing

Impact performance is very important for many applications where the polymeric components are in danger of impact damage. It is quite difficult to evaluate and this is usually done either by devices involving falling objects or by pendulum impact machines which measure the impact strength by determining the energy absorbed during fracture. Charpy and Izod-type tests are common pendulum-based tests and involve the use of carefully prepared specimens with a blunt notch. It has to be noted that the use of the term impact *strength* is rather misleading because it is measured by the energy of unit fracture surface area and has units of J/m^2, not Pa.

Friction and wear resistance

Friction and wear are more complex materials properties. Abrasion/wear resistance in particular, which is directly associated to friction and strength, has great practical significance for durability. High molecular weight, semi-crystalline polymers (e.g. high-density polyethylene (HDPE)) possess toughness which usually provides excellent abrasion resistance. Rubbers generate high frictional forces (coefficient of friction $\mu \approx 1$–3) but they are susceptible to wear and they have to be reinforced with microscopic organic (e.g. carbon black) or inorganic (e.g. silica) particles in order to be used in engineering applications. On the other extreme, polytetrafluoroethylene (PTFE) exhibits very low friction ($\mu \approx 0.03$–0.15). In many cases the friction coefficient is a function of sliding speed. Moreover, Amontons' law – which states that the frictional force does not depend on the apparent area of contact and is proportional to the load – does not hold for all polymers.

Thermal properties

Polymers tend to expand readily as the temperature rises, and for unfilled polymers the linear thermal expansivity, α_l, is generally an order of magnitude larger than for metals and ceramics. The thermal conductivity, λ, can be very low because energy transfer between polymer chains or through polymer networks is inefficient. For this reason polymers are widely used for thermal insulation, especially in fibre or foam form. Specific heat capacity, c_p, does not range widely. **Table 2** provides the thermal properties of some typical polymers.

Permeability

Most polymers are not porous (unless designed with a porous structure) and consequently show excellent barrier

Polymer	α_l: $10^6\,K^{-1}$	λ: W/(m K)	c_p: kJ/(kg K)
PS	70	0.18	1.3
PC	60/70	0.21	1.17
PMMA	70	0.18	1.47
PVC	70/80	0.14/0.17	0.85/0.9
PP	150	0.17/0.22	2.0
HDPE	200	0.38/0.51	2.1/2.7
LDPE	250	0.32/0.40	2.1/2.5
PA 6	80	0.29	1.7
EP	11/35	0.88	0.8
UP	20/40	0.70	1.20

Table 2 Thermal properties of some polymeric materials (adapted from Ehrenstein, 2001)

Polymer	Film thickness: μm	Water vapour: g/(m^2 day)	Air: cm^3/(m^2 day bar)
PS	50	14	80
PC	25	4	–
PVC	40	7.6	28
PP	40	2.1	700
HDPE	40	0.9	754
LDPE	100	1	1100
PA 6	25	80/110	–
PUR	25	13/25	–

Table 3 Indicative values of permeability to water vapour and to air for some common polymers (adapted from Ehrenstein, 2001)

properties to gases, vapours and liquids. However, when polymers are used in the form of thin films and surface coatings, they cannot be considered impermeable. For example, during one day a 0.1 mm thick low-density polyethylene (LDPE) film can transmit ≈ 1 g/m^2 of water vapour. The amount of fluids transmitted depends strongly on the molecular size and chemical composition of the fluid. This is because the permeability depends strongly on the ease of diffusion of the fluid molecules and its interactions within the polymeric material. The polymeric microstructure is also a very important factor as the higher density of crystalline domains inhibits diffusion. In general, the permeability is enhanced with increasing temperature. Plastic tanks and pipes used for water and gas storage and distribution are generally thick enough to withstand high stresses and for this reason they can usually be considered impermeable. However, in some cases of organic fluids (in vapour or liquid form, e.g. hydrocarbons), the attractive interactions at the molecular level can cause increased permeation and consequently contamination of potable water. In **Table 3**, we present the permeability of some common polymers to water vapour and air at room temperature.

Environmental resistance and durability

The environmental resistance which ultimately determines the durability of a component is a significant property of materials for engineering applications. The environmental factors and their usually joined action affecting durability is a complex process. Such factors include: sunlight, oxygen, heat, ozone, ionising radiation, biological organisms, water, solvents and organic fluids. The conjoint action is a crucial point because, for example, oxygen at moderate temperatures or sunlight induces thermal or photooxidation while at high temperatures the result is combustion. Water induces hydrolysis and heat alone produces pyrolysis. The combination of atmospheric oxygen, water and sunlight results in weathering and aging, while solvents and organic fluids induce softening and ultimately dissolution; the effect is exacerbated by heat. Most of these effects are predominantly chemical in nature with the exceptions of softening and dissolution which is physical but in all cases occur at the molecular scale (although obviously the consequences are at all scales). In most cases these changes bring about degradation and dramatic deterioration of mechanical properties and performance. Oxidation may induce cross-linking and brittle fracture or may rupture the chains affecting drastically the microstructure, solubility and the mechanical behaviour including resistance to wear. This affects mainly the surfaces and can have important consequences for the appearance through chalking and loss of surface appearance.

The conjoint action of external and internal stresses can also affect environmental degradation. An important phenomenon is *environmental stress cracking* (ESC) where organic substances (notably surface active chemicals such as detergents) can bring about a slow but brittle cracking in stressed specimens at stresses well below the purely mechanically expected failure stress. It is thought that the chemicals act at the points of local increased stress, producing local mainly physical damage at the molecular scale (plasticisation) which slowly but steadily propagates. PE is a notable example of a polymer which suffers from this mode of failure. Increased molecular weight yields a significant improvement and resistance to ESC. PVC pipes can also suffer from such failure from the action of hydrocarbon impurities. It is also well known that organic solvents can induce unsightly crazes in polymers such as PS. *Crazing* is a characteristic form of damage in glassy polymers in which polymer fibrils extend across slow-growing cracks. ESC enhances the rate of crack growth by softening the material and making fibril extension easier.

However, it has to be noted that contrary to metals and cementitious materials, a few polymers such as PE can be very resistant to strong acids, highly caustic alkalis and aqueous salt solutions. The situation with organic substances is more complex and test data have to be consulted with care on specific polymers which are expected or suspected to come into contact (regularly or incidentally due to failure of equipment) with organic chemicals.

Fire properties

Most polymeric materials are susceptible to relatively easy ignition above a critical temperature which leads to combustion, a rapid oxidation process which often involves the production of a flame. Although all polymers are considered combustible, their fire behaviour is a complex issue and can vary from one polymer to another. The external source of heat which brings about ignition and

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combustion acts usually at the surface and produces organic vapours of low molecular fragments and monomers which in some cases can be toxic. The oxidation process is *exothermic* and produces heat, the rate of which above the ignition temperature is higher than the losses to the environment and the combustion process becomes established and promotes further degradation. The end products of the reaction depend on the polymer composition; for example, in the case of PE, the material completely volatilises and no residue remains, while in the case of PVC a solid carbonaceous char is formed. The amount of heat varies: hydrocarbon polymers such as PE and PS release similar and rather large amounts of heat while oxygen-containing polymers such as polyoxymethylene (POM) release substantially smaller amounts of heat. Polymers such as PVC and PTFE which contain halogens do not burn easily. However, PVC can decompose at rather low temperatures, releasing large amounts of acidic hydrogen chloride. Because of the often poor fire properties of many polymers, it is common to use additives such as mineral fillers like nanoclays which promote flame retardancy.

Biological attack

In contrast to natural polymers (cellulose, casein), most synthetic polymers are not susceptible to microbial micro-organism (bacteria, fungi) attack. In the case of plasticised PVC, it has been shown that the biological attack does not result from the polymeric material but is due to the plasticiser used. Consequently, more polymers withstand soil burial with ease without degradation. Of course, this also has a negative connotation, since due to this resistance polymers cannot biodegrade easily and can be a source of environmental pollution. However, some notable exceptions are some heterochain fibre-forming polymers such as UP and PA, which can suffer in burial tests from some deterioration of mechanical properties which is attributed to microbial attack.

Toxicity

While solid polymers are not usually toxic at normal use temperature, their constituent monomers can be highly toxic and should be handled with care. Furthermore, potentially toxic monomers and other toxic gaseous substances can be released as products of pyrolysis and combustion. Moreover, many low molecular weight additives can present toxicity problems which have to be taken into account when polymers are to be used, e.g. in contact with potable water.

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Chapter 49

Polymer uses in civil engineering

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Construction is one of the largest markets for polymer materials. A huge variety of plastics and rubbers find a multitude of uses. Among the most important are in pipes, geosynthetics, coatings and adhesives.

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Introduction

Polymer materials do not generally compete with the main established metallic and ceramic load-bearing materials but, nonetheless, in recent decades have become indispensible in construction engineering. They offer a great range of valuable material properties, are generally softer in behaviour and able to tolerate large strains. Their distinctive manufacturing technology, using moulding and extrusion processes, opens up new possibilities in product and component design, sometimes strikingly (**Figure 1**). Polymer-based materials have durability and performance attributes which contrast sharply with those of metals and ceramics. They are generally resistant to damage by water, but are prone to air oxidation and have poor fire performance. Here we survey briefly their uses in civil engineering.

Structural plastics and composites

Apart from pipes, large load-bearing components of unreinforced solid polymers are rarely found because of the low stiffness of these materials. Amorphous thermoplastics (PMMA, PC and PVC-U) are used to form roofing and cladding panels and as glazing. Stresses are low and some rigidity can be achieved by means of webs and ribs, or domed profiles. Thermoplastics are widely used to make durable storage tanks up to dimensions of 5 m, particularly for potable water and rainwater. These are most commonly moulded from high-density polyethylene (HDPE), polypropylene (PP), polyvinyl chloride (PVC) or polyvinylidene fluoride (PVDF). Larger tanks and vessels of complex shapes are frequently made of glass-fibre-reinforced plastics (mainly unsaturated polyester materials). Such complex constructions can also be made from thermoplastics by moulding, or extrusion and welding (**Figure 2**). Polymer materials are also used for building panels, often of multi-layer sandwich construction to provide rigidity and thermal insulation.

Ropes and bars

Polymer materials achieve maximum stiffness when drawn to extend and orient the primary macromolecular chains. Some stiffening can also be achieved by stretching in two directions at right angles (*biaxial orientation*). Many semi-crystalline thermoplastics are good fibre formers, notably the polyamides (PA), some polyesters, polyacrylonitrile (PAN) and polypropylene (PP). The tensile modulus can be increased by a factor of 10 or more by fibre-drawing. Such polymer fibres can then be bundled into yarns to form strong ropes. The stiffest and strongest of these polymer fibres are the polyaramids. These are now used as cable stays for bridges and as roof rigging, and as prestressing tendons for concrete. These polyaramid ropes have tensile moduli as high as 120 GPa and strengths of about 2 GPa. The durability of these materials in wet environments brings conspicuous benefits over steel. Polyaramid fibres can provide high-performance reinforcement both in fibre-reinforced polymer composites and also in fibre-reinforced cements. There is some creep and stress relaxation. The viscoelasticity of aramid fibres has recently been characterised (Burgoyne and Alwis, 2008).

Pipework

Large-diameter polymer pipes are used widely for water and gas distribution, drainage and sewerage, and for handling industrial effluents and slurries (Moser, 2008; Sixsmith and Hanselka, 1997; Smith, 2005; Trew *et al.*, 1995). Polymer materials are also used as liners for sewer repair and pipeline renovation. Smaller-diameter pipes are used in rainwater and waste systems, and for water services. Large-diameter systems for water and sewer systems are predominantly of polyethylene HDPE and of PVC, with some use of polyester–glass fibre composite pipes. Smaller-diameter water systems employ a wider variety of materials, including PE, PP, chlorinated PE, PVC (both unplasticised PVC-U and

Figure 1 Tensioned fabric roof: Schlumberger Cambridge Research (courtesy of Simon Glynn)

chlorinated PVC-C), ABS and PVDF. The stiffness and working temperature of HDPE pipe may be increased by chemical or electron-beam crosslinking during or after extrusion. (Cross-linked PE is commonly designated PEX.) Thermoplastic manufacturing methods also allow double-wall or multi-layer pipes to be produced. Materials selection is based primarily on long-term strength and creep resistance, service temperature and durability. Thus PVC-C may be used for hot water services up to 90°C, and ABS can be used at temperatures as low as −40°C. In all cases, special considerations arise when pipe systems are specified for use in contaminated land, as external attack and penetration by chemicals may occur.

Figure 2 Inspection chamber fabricated from polypropylene by extrusion and welding (courtesy of Pipex Ltd)

Polymer materials can also be combined in composite construction with other established pipe materials. For example, steel-in-plastics pipes are used to provide thermally insulated systems for chilled or heated water (BS 7572: 1992, BS EN 253: 2009). A typical construction is a steel service pipe with foamed polyurethane PUR interlayer and a polyethylene HDPE casing. For potable water pipework in contaminated land, a multilayer pipe has been developed in which an aluminium barrier layer is incorporated in HDPE. The aluminium reinforcement also provides improved pressure ratings (Bowman, 1993).

Because of viscoelastic stress relaxation, polymer pipes are tolerant of deflections and deformations caused, for example, by ground movements. However, they are subject to static fatigue. For pipework with sustained internal pressure the long-term stress is an important design consideration. Over the last 30 years, substantial increases in long-term strength have been achieved. For example in polyethylene, long-term strengths have risen from 6 to 10 MPa, highest strengths being provided by the *bimodal* HDPE polyethylene grades. These are *minimum required strengths*, measured from failure data obtained at 20°C under hydrostatic stress and extrapolated to 50 y (BS EN ISO 9080: 2003). PE materials are susceptible to slow crack growth, which is assessed by the *PENT* notch tensile test (BS ISO 16241: 2005), originally developed for large-diameter gas pipes. The PENT failure lifetime has increased by a factor of 10 or more as polyethylene materials have evolved.

Failures in service for polymer pipes directly attributable to materials include environmental stress cracking (ESC) which can be traced to exposure to chemically active substances in the environment or in the piped fluid, often at low concentrations (Wright, 2001). Semi-crystalline thermoplastics (such as the polyolefins) are less susceptible to ESC than amorphous polymers (such as PMMA or PC).

Membranes and geosynthetics

Polymers are readily formed into continuous membranes, sheets, meshes and textiles, the use of which in civil engineering has been an important recent area of innovation (Koerner, 2000; Sarsby, 2000, 2006). Broadly, we can distinguish between simple impervious membranes formed by extrusion of thermoplastics and elastomers and used primarily to contain water as liners, as roofing materials, and as barriers to capillary water movement in floor and wall construction; and thermoplastic textiles and meshes used primarily as soil reinforcement (*geotextiles*). There is also expanding use of polymer materials in tensile and air-supported roofing. These materials are generally based on thermoplastics reinforced with glass-fibre textile meshes.

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Simple polymer sheet is made from high density polyethylene (HDPE), polypropylene (PP), poly(vinyl chloride) (PVC), or elastomers such as chlorosulphonated PE, butyl or EPDM rubber. These sheets may be unreinforced, or reinforced with a textile or mesh. Geotechnical applications are as reservoir, tank, tunnel and landfill liners, for water and for chemical and waste containment (Sarsby, 2000, 2006). Important aspects of specification concern resistance to puncture, tear strength and durability. Below ground, PE and PP have excellent resistance to biological decay, although damage may be caused by animals and insects (as in any polymer membrane). For chemical containment, polymers such as epichlorohydrin rubbers (ECO) are selected. These have good barrier properties against diffusive transport of non-aqueous chemicals (such as oils and gases). It is usually necessary for continuous sheets (which may be as wide as 10 m) to be joined on site by welding or, occasionally, by adhesive bonding.

Loose-fitting tubular polyethylene (PE) film has a well-established and effective use to protect buried ductile iron pipe from corrosion (BS 6076: 1966).

Geosynthetics (geotextiles, geogrids and geomembranes) are now widely used for ground stabilisation, separation and for drainage and filtration (Carter and Dixon, 1995). These are commonly made of polyethylene, polyester and polypropylene, in numerous forms and with controlled strength and permeability. For example, PP and PETP geogrids are used as base reinforcement in road construction. Geosynthetics find similar uses in rail track construction to stabilise and separate ballast.

Fabric structures provide a good means of enclosing or roofing large areas (Huntington, 2004). The first tension roof structures were built in the 1970s. The original material, a woven glass-fibre textile coated in polytetrafluorethylene PTFE, has now demonstrated its durability in that application over more than 30 years. The material may equally be regarded as a glass-fibre-reinforced PTFE membrane. Other less durable roofing membrane materials are all-polymer PVC-coated polyester, and various silicone and polyolefin fabrics. PTFE fabrics for tensile structures have a plain weave of glass-fibre yarn (the yarn having about 200 filaments, each 3.8 μm in diameter) coated with PTFE to produce an impermeable fabric 0.8–1.0 mm thick. The mechanical properties of PTFE glass-fibre materials are largely controlled by the glass textile, and are strongly directional since the textile is composed of fibre arrays at right angles. Full biaxial testing is required to determine the complex non-linear stress–strain behaviour (Bridgens *et al*., 2004).

Coatings

Surface coatings (*paints*) are widely used in the construction industry to protect timber surfaces and to prevent or reduce the corrosion of metals. Coatings may be applied on site or during manufacture or fabrication. In almost all these applications, polymer materials provide the coating film and bind the functional or decorative pigments which may be present. If the film is unpigmented and more or less transparent, the coating is often described as a varnish.

A great variety of polymers are now used in the formulation of paints and coatings (Lambourne and Strivens, 1999; Bentley and Turner, 1997; Stoye and Freitag, 1998). We can base a broad classification of coating types on the mechanism of film formation or drying. In some coatings (such as chlorinated rubber (CR), PVC and bituminous paints) the polymer is dissolved in a volatile solvent, and the applied coating dries simply by solvent evaporation. There have been strong environmental and safety pressures in recent years to reduce the quantities of volatile organic solvents released in painting operations. However, polymers used in coatings are insoluble in water (as they must be for performance reasons), so that water cannot be used directly as a solvent. Water can, however, be used a dispersing medium for fine particles of polymers to make *emulsion* or *latex* paints. Emulsion paints dry by evaporation of water followed by coalescence of the polymer particles to form a continuous film. This process is irreversible and the polymer binder does not redisperse in contact with water.

Other paints dry by chemical reaction, either with oxygen from the air or between components of the paint. Thus traditional oil or *oleoresinous* paints lose solvent by evaporation but then absorb oxygen which forms cross-links between polymer chains to create a three-dimensional network within the coating. Other *air-drying* paints contain alkyd, urethane alkyd or epoxy ester binders. An important group in which film formation is by reaction between two pre-mixed components are the paints based on two-pack epoxy binders. These dry and cure by the same mechanism as epoxy adhesives. Two-pack polyurethane paints also dry and cure chemically.

Some coatings, primarily applied during manufacture, are formed from a dry powder which is deposited (usually electrostatically) and subsequently melted to form and cure a continuous film. Powder coatings may be based on thermosets (such as polyesters or epoxies) or thermoplastics (for example acrylics). Powder processes use no solvent and produce tough, thick high-build coatings (Grainger and Blunt, 1998).

For steel structures, BS 5493: 1977 and its EN successor BS EN ISO 12944-5: 2007 provide technical guidance. Protective paint systems invariably comprise two or more coats, including a primer applied directly to the prepared metal surface, usually containing zinc powder as a metallic pigment, **Figure 3**. A typical protective paint system for low-alloy carbon steel might consist of a zinc-rich two-pack epoxy primer, followed by two coats of chlorinated

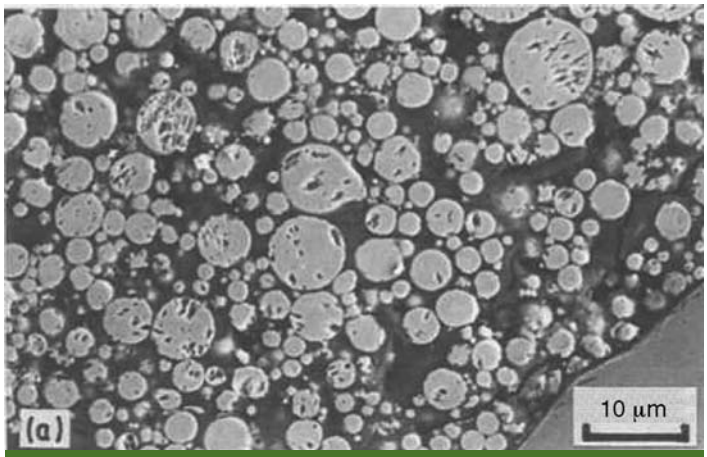

Figure 3 A section through a zinc-rich coating for corrosion protection, showing high concentration of zinc metal particles in the binder (reprinted from M. Morcillo, Journal of Materials Science ©1990 Springer)

rubber paint to form a finished dry film 160 μm thick. The overall paint system provides electrochemical (cathodic) protection through the zinc metal particles and a barrier to water and oxygen ingress. Epoxy and PVC coatings are also used to protect steel reinforcement against corrosion when concrete is used in harsh environments (Andrade *et al.*, 1992; ISO 14656, 1999).

Adhesives and sealants

Polymers are the basis of all engineering adhesives (Adams *et al.*, 1997; Institution of Structural Engineers, 1999; Mays and Hutchinson, 1992; Packham, 2005). High-performance adhesives are formulated to set and develop strength by chemical reactions and so are thermoset materials. There are therefore similarities between adhesives and the matrix component of composite reinforced plastics (and indeed also between adhesives and paints). Thus structural *glulam* laminated timber (or indeed plywood) may be regarded as an adhesive bonded wood composite.

Epoxies are especially versatile and can be formulated for application on many substrates. Other structural adhesives are based on polyester, polyurethane, acrylic and cyano-acrylate polymers. For laminated timber and other wood products, such as particle board made under factory conditions, the long-established adhesives based on urea-formaldehyde (UF) (*aminoplastic*), phenol-formaldehyde (PF), or phenol-resorcinol-formaldehyde resin systems are widely used. These resins provide excellent adhesion to timber and proven performance (Dunky, 1998). UF has poor water resistance and the phenolic PF and phenol-resorcinol-formaldhehyde adhesives are used for exterior or otherwise severe conditions.

There are also thermoplastic adhesives which set simply by solvent evaporation. These are the basis of well-known commodity adhesives which are formulated from polystyrene or elastomer polymers. These do not develop high strength but are convenient one-pack materials, which are easy to apply and can provide rapid bonding as impact adhesives. Closely related to the simple solvent adhesives are the latex adhesives in which the polymer binder is present as a polymer-in-water emulsion. The most important of these are the general-purpose latex wood adhesives containing poly(vinyl acetate) PVAC or its copolymers as the main component. They lack wet strength and are not sufficiently strong under sustained load to be regarded as structural adhesives.

We mention briefly that another subset of polymer materials finds applications as gap-filling sealants, necessary in much modern construction which needs to shed rainwater in large quantities. These materials are formulated to resist shrinkage, to be durable and to provide good bond to a variety of relatively unprepared surfaces. Construction sealants (type F) are classified (BS EN 11600: 2003) according to their ability to accommodate movement (glazing sealants form a separate group, type G). These sealants are based on a variety of polymers, most commonly elastomers such as polychloroprene (CR) and nitrile rubber (NBR), silicones (SI) ('Q' group elastomers), polysulphides, polyurethanes and acrylics. Some of these sealants cure by reaction with moisture or ultra-violet light.

Expansion bearings and anti-vibration mounts

A minor but technically demanding use of polymers is in expansion bearings for bridge and pipeline construction. These may be fabricated either with a durable synthetic rubber such as polychloroprene CR (neoprene) or with a low-friction thermoplastic, usually polytetrafluorethylene (PTFE). In the sliding bearing, the PTFE pad, typically 5 mm thick, is bonded to a steel substrate plate. In use, it slides on a polished stainless steel countersurface, on which it has low friction (friction coefficient about 0.05). For high loads, a stiffer glass-filled PTFE material may be substituted.

A related specialised application of elastomers is in anti-vibration mounts for control of vibration and shock in structures, for example generated by pumps, fans and other machinery. Natural rubber is commonly chosen for this function.

Chemical grouts for soils

Polymers based mainly on water-soluble acrylamide monomer have played a minor but long-established role in the stabilisation of soils or in reducing water permeability (Karol, 2009). To a large extent this technology has been superseded by the use of geosynthetics. Chemical grouts have also been used to control water infiltration in sewers

and manholes. Acrylamide is dissolved in water and mixed with a cross-linking agent and a chemical retarder. The low-viscosity mixed fluid is injected into the unstable ground, and after reaching its target it forms a weak bonding gel as the retarded cross-linking reaction progresses. There have been health concerns over the handling of acrylamide or other monomers, and the possible presence of unreacted acrylamide in grout.

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Section 7: Polymer fibre composites in civil engineering

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Chapter 50

Polymer fibre composites: an introduction

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In recent years there has been an increasing interest in the use of fibre-reinforced polymers (FRP) in the civil engineering industry throughout the world. Specifically from a UK perspective, but also from a worldwide point of view, the involvement with this material is largely as a result of innovations in structural and civil engineering FRP systems. This interest commenced after the surveys of 1974 and 1978, undertaken by the National Economic Development Office, which reported on several poorly performing projects and the low-technology, low-skill and labour-intensive civil engineering industry. In response to this, the government commissioned Sir M. Latham to lead a year-long investigation, which was sponsored by industry; later Sir J. Egan produced a further report. The main tenets of the reports were that the industry needed practical steps to improve productivity and profits and decrease quality failings and accidents. This chapter discusses the results of this investigation, the development of fibre–matrix composites over the past three decades and the areas where the FRP material is currently used in construction.

Introduction

The lack of investment in research and development in construction over the 40 years following World War II meant that methods and material in the construction industry had changed little over that period. It is therefore not surprising that the plastics/composites industry did not show interest in the civil engineering industry where the possibility of utilising new materials and therefore market opportunities were not visible to potential investors. The technological revolution in materials and processing in all other sectors of the manufacturing industry had largely bypassed the construction industry. Furthermore, the advent of the 1990s saw a widespread increase in concern for 'value for money'. This was particularly true for the UK construction industry, which had a long history of failing to satisfy the expectations of many of its clients. In 1974 the National Economic Development Office suggested that nearly one in five clients were dissatisfied with the service they had received from the industry; similar results were again obtained from their 1978 survey. In response to this situation the UK government commissioned Sir Michael Latham, a former Conservative MP and ex-director of the UK Housebuilders' Federation, to lead a year-long investigation. The Latham Report was produced in 1994 and, in addition to being sponsored by the government, it was also sponsored by industry, following several poorly performing projects (Latham, 1994). The inefficiencies identified pointed to the need for greater partnering and collaboration in the construction sector.

In the late 1990s, with a fresh change in government thinking, Sir John Egan was commissioned by the Office of the Deputy Prime Minister to produce a report on 'Rethinking Construction'. A Construction Task Force was set up and their report was published in 1998 (Egan, 1998). The main tenets of the report were that the industry needed practical steps to improve productivity and profits, and to decrease quality failings and accidents.

The Construction Industry Board (CIB) was established in 1995 to improve the performance of the UK construction industry. It brought together both suppliers and customers from the private and the public sectors with central government; its inauguration followed Latham's report in 1994. The findings and conclusions of the Latham and the Egan Reports are mirrored in the Report of the Construction Review Committee published in 2001. In the 1990s it was clear that the construction industry was regarded as low technology, low skill and labour intensive compared with most other industries. The mismatch between research investment and construction expenditure meant that construction had proceeded on a scale with an inadequate understanding of many aspects such as durability deterioration mechanism for structures, and this had often meant that due allowance had not been made for practical repair and maintenance. However, notwithstanding the above reports, there have been encouraging signs that within the past 20 years the transition from the conventional materials to the more advanced materials is being spearheaded by the construction industry, where 30% of all polymers produced are now utilised in that industry. The rapidly growing interest in composites, made from fibre-reinforced polymers, and used in specific areas of the construction industry

is fuelled by the need to improve durability, to reduce site labour costs and construction time and to improve safety. The UK fibre-reinforced polymer (FRP) composite industry produces 240 000 t of FRP products a year, with 11% of this being used in the construction industry (Conroy *et al.*, 2005). This current situation has resulted from the utilisation of the research and development findings into new fibres and matrix materials, and automated fabrication processes relevant to applications in the construction industry. Large investments have been made by the US government in conjunction with private industry in the *High Performance Construction Materials and Systems*. The proposed materials include *advanced polymer composites* and systems in which these materials are employed, including new methods of construction and robotics. Governments' initiatives in Europe have been slower than those in the US. The UK government launched two initiatives with similar objectives to those in the US, under the control of the Engineering and Physical Sciences Research Council (EPSRC); these were the '*Innovative Manufacturing Initiative 1994*' and the '*Materials for Better Construction Programme*' also 1994. There have been no similar initiatives in civil engineering launched by the government since that time; the interest has passed mainly to buildings and some low-grade indirect interests to the civil engineer. The main thrust currently is the formation of technology centres such as the Advanced Composites Centre for Innovation and Science (ACCIS) at Bristol University, UK, which incorporates four central themes: multifunctional composites and novel microstructures; design, analysis and failure; intelligent structures; and composite processing and characterisation. This centre includes the Rolls-Royce University Technology Centre (UTC) and the Smiths Aerospace-EPSRC, but these latter centres, as the titles suggest, are mainly related to the aircraft industry.

The report, HC 87 Session 2000–2001 dated 11 January 2001, by the Comptroller and Audit General of the National Audit Office states that:

> *Research and development in the construction industry is low. In 1999/2000 in an industry with a turnover of £65 billion, £270 million was spent on construction research and development, of this £47 million was funded by central government departments; £147 million by industry and the remainder by other organisations (Research Councils, Funding Councils, European Union). However, the majority of the industrial research is commissioned by the construction materials, components and systems suppliers. The ability of the industry to improve and to be innovative could be impeded if it does not invest more in research and development.*

There is now a much greater awareness of the importance of design in relation to product performance and reliability. However, there is only a limited number of standard specifications and codes of practice in civil and structural engineering applications of composites, and therefore the vitally important items of reliability, performance and durability are difficult to address, unless through specific proven systems.

The development of fibre–matrix composites in construction

Most natural materials consist of two or more substances; when these substances are combined together a material is produced which is able to perform its required function more efficiently than the single substance by itself. These materials are known as composites. Bone, for instance, achieves its combination of lightness and strength by combining crystals of apatite (a compound of calcium) with fibres of the protein collagen. Wood, the most widely used structural material from earliest times, is an example of a naturally occurring composite material. A man-made composite for civil engineering use is reinforced concrete. The fibre–matrix composite is a later addition to the range of composite materials used by the civil engineer.

The range of fibre/matrix composites is very diverse but carbon, glass and aramid fibres are the ones mainly used in civil engineering in combination with polyester, vinylester and epoxy polymers to form the advanced fibre–polymer composite for structural members. There are other fibres used in construction mainly for geosynthetic applications; these fibres are manufactured from the thermoplastic polymer; this topic is discussed in Section 6.

Section 7 of this book first discusses the types of thermoplastic and thermosetting polymers and their mechanical and in-service properties. It then discusses the types of fibres and their mechanical properties that are used to reinforce the polymer, and introduce the manufacturing procedures available to combine the polymer and fibre materials into structural components. The composite materials are manufactured by controlled distribution of the reinforcement (phase 1), in a continuous phase of a second, the matrix (phase 2). The boundary between the matrix and the reinforcement, the interface (phase 3), is controlled to obtain the desired properties from a given pair of materials. When dissipation of impact energy is required, the interface may be made weaker by minimising the chemical coupling of the reinforcement to the matrix. However, it is much more likely that the coupling between the two phases (1 and 2) is maximised. This interfacial coupling will then allow stresses, dispersed through the matrix, to be transferred to the reinforcement. Coupling is provided by wetting the reinforcement by the matrix in molten or low-viscosity state. The matrix plays several important roles in the overall composite characteristics. It bonds the reinforcement together in the correct design configuration, thus protecting the reinforcement against abrasion or environmental corrosion. The load carried by the composite is distributed to the fibres by way of the

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matrix. To be able to transfer the loads and reduce the chance of failure in the matrix, adhesion to the reinforcement must be coupled with sufficient matrix shear strength which, in general, is proportional to the tensile strength. However, high-strength matrices tend to be brittle and when high tensile and flexural properties are provided by strong and stiff but brittle reinforcement, fracture toughness is provided by the plastic flow at crack tips in the matrix, which absorbs energy and reduces stress concentration. Plastically deforming matrices also deflect cracks parallel to fibres in one plane. In addition to the two phases there is the boundary between the matrix and the reinforcement, known as the interface area (phase 3), which controls the interaction between phases 1 and 2. The continuous matrix material is formed from two components – the resin and the curing agent (hardener) – these are mixed in specific proportions depending upon the resin and time required for initial cure to take place.

Advanced fibre–polymer composite materials used in civil engineering are invariably referred to as fibre-reinforced polymer composites (FRP) and this term is used in all subsequent sub-sections of Section 7. There are two types of curing agents for the resin material: the cold and the hot cure methods. The first uses a cold cure resin and the second a hot cure resin. The cold cure procedure is undertaken in situ in the field, (i) to impregnate dry fibres to form the FRP composite, or (ii) to produce an adhesive. The environmental temperature of the site would determine the cure temperature and likewise the length of the polymerisation period. It is advisable when using a cold cure resin to post-cure the material at an elevated temperature, the length of the cure period will depend upon the value of the elevated temperature; the higher the post-cure temperature the shorter will be the post-cure period. The hot cure procedure (for the hot cure resin) is used for the manufacture of structural members by automated techniques such as the pultrusion or the fibre pre-impregnation; the polymerisation of the hot cured resins is undertaken in a factory environment under controlled conditions of temperature and pressure. The manufacturing techniques of composites and the polymerisation of the polymer procedures will be discussed at length in the *Characterisation of fibre and matrix materials used in construction* and *Advanced polymer composites used in construction*. The testing and evaluation of polymers and polymer composite materials are presented in *Failure criteria of composites*.

Bonding polymer composites onto similar and dissimilar materials will be discussed in *Codes and standards for composites*. This area is particularly important when retrofitting a structural member damaged in a seismic event or when rehabilitating a structural member weakened through degradation or to accommodate a structural member that is being upgraded due to a change in use of a building resulting in greater imposed live load. To upgrade a member using advanced composites, a plate is made of this material and bonded to the structural member using a cold-cured adhesive polymer. The Achilles' heel in this form of construction is the cold-cured adhesive due to possible insufficient curing of the polymer and resulting in its poor adhesion and low glass transition temperature. An alternative rehabilitation technique is the use of a hot-melt pre-impregnated fibre which is cured on site in conjunction with a compatible film adhesive. The polymerisation of the polymer procedure is undertaken at an elevated temperature and under a vacuum-assisted pressure of 1 bar. These methods will be discussed in *Advanced polymer composites used in construction* and *Applications*.

Advanced polymer composites are utilised in a number of ways in the construction industry but arguably their greatest use, currently, is in the rehabilitation and retrofitting of reinforced and prestressed concrete (RC and PC respectively), metallic, masonry and timber structural members. External FRP plates are bonded to the weakened part of the structural beam or near-surface-mounted rods are bonded into grooves cut in the concrete beam. FRP composites are also used in a number of other areas in construction as FRP rebar reinforcement for RC, prestressed and post-tensioned tendons. Complete bridge structures have been manufactured using FRP composites. Chapter 9 discusses some of these uses.

Thermosetting polymers are one of the least recyclable materials due to the cross-linking that takes place during the curing reaction which produces covalent bonds between the long polymer molecules. The resulting chemical structure is a hard material and when heated to about 400°C will disintegrate chemically and will burn if oxygen is present. In addition, to meet particular property specifications, thermosetting polymers will be modified by the addition of different additives and fillers, which then form a complex material. Thermosetting polymers cannot be depolymerised readily to regain their molecular building blocks. With the introduction of the new European waste directives on land fill and incineration, pressure will be put onto the FRP industry to solve their waste through recycling and reuse. Currently, about 70% of FRP composite waste comes from the end-of-life of a component and the remainder of waste comes from production waste. The recycling of FRP material in civil engineering is discussed in *Recycling of FRP material in construction*.

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Chapter 51

Characterisation of fibre and matrix materials used in construction

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This chapter introduces the components of fibre-reinforced polymer composites and their methods of manufacture; it discusses the advantages associated with their mechanical, physical and in-service properties as well as their short-comings. There are three specific fibres used in the civil engineering industry, namely, the carbon, the aramid and the glass. Each fibre has its unique mechanical and in-service properties. In addition, each of the three types of fibre contains several variations in chemical formulations, giving each their own specific properties. A brief description of the method of production for the three fibres is given. Furthermore, the chapter discusses the three types of polymers that are used in construction: namely the polyester, vinylester and the epoxy, each of which has specific functions in construction. The elastomeric polymer is also discussed; this material is used in bridge-bearing applications or as vibration isolators under buildings. These elastomeric materials are generally laminated in conjunction with steel plates. A brief discussion relating to the main synthetics used in civil engineering is given.

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Introduction

The reinforcement of a relatively low-modulus polymer with a high-modulus, high-strength fibre uses the plastic flow of the polymer material under stress to transfer the load to the fibre. This produces a two-phase material in which the primary phase (the fibre) is well dispersed in and bonded by the weak secondary phase (the polymer). The principal constituents of the composite are therefore the fibre, the matrix (the polymer) and the interface. This last component is an anisotropic transition region with a graduation of properties. The interface is required to provide adequate chemical and physical bonding stability between the fibre and the matrix in order to maximise the coupling between the two phases and thus allow stresses to be dispersed through the matrix, and thence to be transferred to the reinforcement. By wetting the reinforcement with the matrix in the molten or low-viscosity state, coupling between the two components is provided.

The matrices for fibre-reinforced polymer composites

Introduction

A matrix (polymer) is formed by the chemical combination of identical units with large molecular mass composed of repeating structural units or monomers connected by covalent chemical bonds. The process by which polymers are formed is known as polymerisation, which occurs as a result of addition reaction or condensation reaction. There are many polymers on the market and the topic of polymer chemistry is a very wide subject with much research and innovation taking place in this area.

The characterisation of a polymer requires several parameters which need to be specified. A polymer consists of a statistical distribution of chains of varying lengths and each chain consists of monomer residues which affect its properties.

The polymer is required to fulfil the following functions:

- to bind together the fibres and protect their surfaces from abrasion and environmental corrosion;
- to wet out the fibre and to cure satisfactorily in the manufacturing process;
- to disperse the fibres and separate them in order to avoid any catastrophic propagation of cracks;
- to transfer stresses to the fibres efficiently by adhesion and/or friction and, in addition, to reduce the chance of failure in the matrix, adhesion to the fibres must be coupled with sufficient matrix shear strength which is, in general, proportional to the tensile strength;
- to be thermally compatible with the fibres;
- to be chemically compatible with the fibres over long periods of time;
- to have appropriate fire resistance and limit smoke propagation;
- to provide finish colour and surface finish for connections.

The matrix polymer material is composed of two components: the resin and curing agent (hardener). In civil engineering construction there are two types of curing

procedures, one using a cold-cure resin and the other using a hot-cure resin. The cold-cure system is generally used on site and the hot curing system is used for manufacturing composite components in a factory under controlled conditions.

There are several different polymer matrices which can be utilised to form advanced composites; within the composite family there are two major types: the thermoplastic and thermosetting polymers. Both types are composed of long chain molecules made by connecting many smaller molecules.

The thermoplastic polymer

Thermoplastic polymers are composed of long chain molecules held together by relatively weak Van der Waals' forces. However, the chemical valency bond along the chains is extremely strong, and their strength and stiffness is derived from the very high molecular weight and the inherent properties of the monomer units. These polymers are either amorphous, which implies a random structure with a high concentration of molecular entanglement, or they are crystalline with a high degree of molecular order or alignment. In the amorphous polymer, the random structure becomes disentangled during heating and changes the material from a solid to a viscous liquid, whereas heating the crystalline polymer changes it to an amorphous viscous liquid. However, it is difficult to make a polymer which has a pure crystalline structure because of the complex physical nature of the molecular chains, consequently, the so-called 'crystalline' polymer should more correctly be described as semi-crystalline.

In construction, the thermoplastic polymers such as the polyolefins and polyesters are used in geosynthetics. The polyacrylonitrile in the form of fibres are used as the precursor for the manufacture of carbon fibre.

The polyolefin and polyacrylonitrile polymers

The polyolefin and polyester polymers are the main thermoplastic polymers used in civil engineering. The material science of these polymers has been discussed in the section on the aramid fibre and their geotechnical applications in civil engineering have been discussed in Sarsby (2006).

Generally, when the thermoplastics polymers are placed under a tensile load, they will behave visco-elastically with a gradually decreasing slope of the stress–strain relationship. At a limiting load value, the material will be unable to support further load increase but will continue to elongate excessively under this load; the slope of the stress–strain relationship will then become zero. During this elongation, the molecules will align themselves with the direction of the applied load forming a 'fibre' in this direction: the greater the time of application of the load the greater will be the elongation and strength of the 'fibre'. The thermoplastic fibres which are utilised in civil engineering are the polyolefin for geosynthetic applications Sarsby (2006), and the polyacrylonitrile (PAN) fibre which is used as the precursor for the manufacture of carbon fibres.

Elastomeric polymer

The elastomeric material consists of very long chain molecules which are twisted and coiled in a random manner and, as a result of this molecular flexibility, they can undergo very large deformations and therefore their modulus of elasticity is low. Being readily stretched is of little value to the civil engineer when required for an elastomeric application, unless the material can return to its original size and shape once the stress is released.

Elastomers are predominantly thermosetting polymers, which require vulcanising. They can also be manufactured from the thermoplastic polymers, but these elastomers are not normally used in construction. The principal difference between these two types of elastomers is the type of cross-linking bond in their structure. The cross-linking of the thermosetting elastomeric polymers takes place when a covalent bond is formed; it is a critical structural factor contributing to the imparting of high elastic properties. The cross-linking is an irreversible chemical reaction, involving the addition of sulphur compounds usually applied at high temperature; the process is known as vulcanisation and is a similar chemical procedure to the curing of the thermosetting polymer. The vulcanisation procedure ensures that the elastomer will return to its original configuration when the stress is removed. In this state the polymer is neither strong nor tough; see the section on in-service properties of the thermosetting polymer, which discusses toughness. **Figure 1** illustrates the difference between a strong but low-toughness material (fibre) and a tough but lower-strength material (composite) and an elastomeric material (elastomeric polymer).

The material science of these polymers is discussed later in this chapter.

The elastomeric polymer is used in bridge engineering as elastomeric bridge bearings and as resilient seatings in structural buildings; further discussions are given later in this chapter.

Thermosetting polymer

The material science of the thermosetting polymers has been discussed. At the completion of the polymerisation (curing) process the thermosetting polymer will have undergone a chemical change in which the molecules of the material will have produced a tightly bound 3D network of polymer chains. The degree of cross-linking of the polymer, the length and the density of the molecular units are a function of the chemicals used in the manufacture of the polymer; in addition, the cross-linking is a function of the degree of cure of the polymer. The molecular units will influence the mechanical properties of the material.

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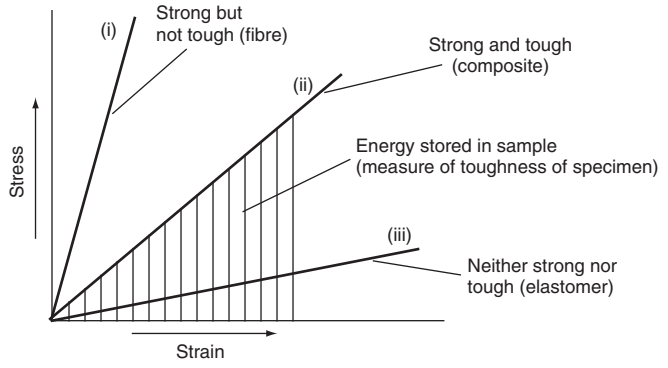

Figure 1 Stress–strain for three different types of strength/toughness material

There are three types of thermosetting matrices used in construction; these are the polyester, vinylester and epoxy, all of which can be reinforced with glass, aramid or carbon fibres to form the fibre–matrix composite for civil/structural utilisations. These polymers are usually made from liquid or semi-solid precursors which cure irreversibly, and on completion of the cure process a hard solid is formed by chemical cross-linking which produces a tightly bound 3D network of polymer chains.

Unsaturated polyesters

Polyester is the simplest of the thermosets and is generally made by reacting dibasic acids such as maleic anhydride or phthalic anhydride with dihydric alcohols (e.g. ethylene glycol) in approximately equal amounts. The resulting polymer is a short-chain polymer with a molecular weight of about 5000 (about 35–40 repeat units or mers) and is a stable liquid. The liquid resin is set into an amorphous solid by cross-linking the polymer chains to each other. The cross-linking occurs by the addition of a small monomer molecule such as styrene. The monomer, like the active sites on the polyester chain has an unsaturated C=C bond and it is this that provides the bridge between the polyester chains. The monomer is usually mixed with the polyester to form the resin. The two components will react but at room temperature the reaction may take years and needs to be initiated by the addition of a catalyst (hardener) such as methyl ethyl ketone epoxide at about 0.5 to 1% by volume. Once the reaction is initiated, the cross-linking generates sufficient heat to further catalyse the reaction and the resin gels (within about 10 min), the cross-linking continues and strength is generated after about 24 hours.

In the civil engineering industry, polyesters are currently not widely used for the manufacture of 'all-polymer composite' structural components, but on occasions semi-structural units have been produced by the automated pultrusion components. They can be readily processed and cured at ambient temperatures using a wet lay-up procedure (see chapter on advanced polymer composites used in construction). For structural applications the vinylester polymers have largely replaced the polyester polymer, which during polymerisation undergoes a large shrinkage, particularly if styrene in monomeric form is used as a reactive diluent, and this causes some dimensional inaccuracy in the geometric shape being formed. These ambient cured polymers must be post-cured to provide long-term durability. Styrene, in amounts of up to 50%, is the most commonly used diluent. However, it should be noted that, during the wet lay-up (hand-laminated) production method, large amounts of styrene and trace quantities of styrene-7,8-oxide (SO) are released, which will cause a health risk unless safety precautions are taken. There are three types of commonly used polyester resins, these are: (a) the orthophthalic which have low thermal stability and chemical resistance; (b) the isophthalic which contain isophthalic acids as an essential ingredient and are of superior quality with better thermal and chemical properties; and (c) the bisphenol A, which are of superior quality than the orthophthalic and isophthalic resins with a higher degree of chemical, thermal and hardness resistance and have some degree of flame resistance. The high shrinkage at cure of the polyester resin mentioned above will cause (i) an increase in hydrophobicity, thereby effectively decreasing the level of moisture absorption and (ii) an increase in shrinkage to levels of 5–19% by volume. This can result in significant micro-cracking in resin-rich areas and high residual stresses in composites having high fibre volume fractions.

Vinylesters

The vinylester is a hybrid form of polyester resin which has been toughened with epoxy molecules within the main molecular structure. Its molecule features fewer ester groups compared to the polyester resin and, as these ester groups are susceptible to water degradation by hydrolysis, vinylesters exhibit better resistance to water and many other chemicals than do their polyester counterparts. Furthermore, as vinylesters have unsaturated esters of epoxy resins, they have similar mechanical and in-service properties to those of the epoxies, but they have similar processing techniques to those of the polyesters. Generally, the vinylesters have:

1. A processing method at both room (cold-cure resins) and elevated temperatures (hot-cure resins); the same warning, regarding the T_g, is given to the use of this polymer as that stated in the first item of the epoxy polymer.
2. Good wetting characteristics and will consequently bond well to glass fibres.
3. Good resistance to strong acids and strong alkalis.
4. Enhanced chemical resistance, compared with those of the polyester as well as reduced water absorption and shrinkage characteristics.

Epoxies

The highest-performance resins currently available to the civil engineer are the epoxy resins; they generally outperform most other resin types in terms of mechanical properties and resistance to environmental degradation. This has led to their extensive use in the construction industry where the most important epoxy resins are the low molecular weight polymers (oligomers), which are produced from the reaction of bisphenol A and epichlorohydrin in an alkaline solution which absorbs the HCl released during the condensation polymerisation reaction. They range from the medium-viscosity liquids through to high melting solids.

There are two basic procedures which are used to polymerise the epoxy polymer for use in the civil engineering industry; these are the cold-cure systems that are cured at ambient temperatures and the hot cure polymers where the polymerisation is performed at elevated temperatures. Thus different resin and curing systems are required for site and factory manufacture of fibre-reinforced polymer (FRP) (viz. the cold-cure systems for site work and the hot cure systems for factory fabrication). Consideration must be given to the *glass transition temperature* (T_g) of cold cure epoxy polymer (see the section under the physical properties of thermosetting polymers). Epoxies used in the construction industry:

1. must possess dimensional stability and high specific strengths;
2. operate at higher temperatures to those of the polyester resins and also possess superior toughness;
3. have low shrinkage during polymerisation and also have good adhesion to many substrates. This allows mouldings to be manufactured of high quality and with good dimensional tolerance;
4. are the hot-cured polymer systems which have a high temperature resistance and can be used at temperatures up to 200°C. Some hot cure epoxies have a maximum temperature range up to 316°C. Ambient-cured epoxy systems are used in the wet lay-up technique and both hot- and cold-cured systems are used as adhesive polymers.

It is essential that the correct mix ratio is obtained between the epoxy resin and the curing agent to ensure that a complete reaction does take place as the amine molecules 'co-react' with the epoxy molecules in a fixed ratio. If the mix is not in the correct proportions, unreacted resin or curing agent will remain within the matrix, and this will affect the final properties after cure. For the cold-cure polymers, manufacturers usually formulate the two component parts, either by weight or by volume, in separate containers which are then mixed to give the correct ratio of the two components; this is particularly useful for site work. The resin container would be of sufficient volume to pour the curing agent into it and to mix by mechanical paddle.

The polymerisation process

The polymerisation process is particularly important for the understanding of the mechanism of the manufacture of polymers and polymer composites. The various methods for the manufacture of civil engineering composites are discussed in Chapter 52 and, depending upon the type of manufacture, either the cold-cured or the hot-cured resins are used. If the composite unit is manufactured by an automated factory production method under a controlled temperature and humidity environment, the hot-cured resin is used. If, however, the composite material is manually fabricated in the field, or if the adhesive polymer is employed on site, the cold-cure systems would generally be used under ambient temperature conditions. The curing procedure for polymers is important as the long-term performance properties of the composite and its durability will be influenced by the degree of cure of the resin, particularly for site-cured systems. The operating temperatures of the manufacturing processes will generally vary between 120°C and 135°C; the actual value will depend upon which system is being used.

All polymers after the initial cure should be post-cured at an elevated temperature for a certain length of time; the higher the temperature of initial cure the shorter will be the post-cure period. However, the hot-cured polymers are not generally post-cured due to the factory controlled method of production, but the initial cure of the cold-cured polymers will reach only about 90% of the full polymerisation value after some 10 days (depending upon the polymer system and ambient cure temperature) and these polymers should be post-cured. The post-cure effects on E-glass/vinyl-ester fibre-reinforced polymer composites manufactured using the vacuum-assisted resin transfer moulding (VARTM) method (see Chapter 52) was investigated by Cain *et al.* (2006). The composites were exposed to varying levels of post-cure temperature and duration, and examined for the effects of cure at various time intervals after post-cure. During this time the matrix polymer was inspected, using Fourier transform infrared spectroscopy to directly assess the degree of conversion. The results suggested that the degree of conversion is limited to 80% for the vinylester oligomer and 90–95% for styrene after a post-cure temperature of 93°C. It was observed that after 300 days of ambient temperature storage, the non-post-cured samples approached the degree of conversion exhibited by those post-cured at a temperature of 93°C. It was concluded that the resin quasi-static properties are greatly affected by the degree of cure; whereas fibre-dominated properties are not affected as much. As mentioned above, this under-cure will affect the long-term mechanical property of the

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polymer with the result that its exposure to a variety of adverse and sometimes harsh environmental conditions met in construction will degrade the polymer and the FRP composite material, and thus will alter their mechanical performance. A further concern regarding the ambient-cured systems is that their relatively low cure temperature will affect their glass transition temperature (T_g), which will generally be about 15°C to 20°C above the cure temperature. This point is discussed in the section on 'Dimensional Stability'.

The physical properties of thermosetting polymers

High-performance thermosetting resins are designed to provide specific properties in highly demanding environments. These matrices usually possess high dimensional stability at elevated temperatures and excellent thermal resistance, low water absorption, good chemical resistance, high mechanical strength, excellent stiffness and high compressive strength. This combination of properties is useful for advanced composites. However, owing to the increase in cross-linking density observed during polymerisation, conventional thermosetting matrices are considered to be brittle and this limits the damage tolerance of the composite (Hamerton, 1996).

Temperature

The effects of temperature on polymers can be separated into short-term and long-term effects. Short-term effects are generally physical and reversible when the temperature returns to ambient, while long-term effects are generally dominated by chemical change and are not reversible. The long-term effects at elevated temperatures are usually referred to as the results of ageing. All physical properties of polymers change as temperature is varied, some to a greater extent than others. As polymer properties are temperature dependent, it will be clear that, to fully characterise a material, properties should be measured over a range of temperatures. These remarks on selection of properties apply equally to measuring the ageing effects of long-term exposure. Certain types of tests to measure short-term effects, such as glass transition temperature, thermal expansion and melting point, are thought of as separate properties although they are particular cases of the effects of temperature.

Thermal analysis

Thermal analysis is the study of one or more properties of a material as a function of temperature, and to undertake this study a thermal analyser is used; it scans property change over a wide temperature range. The instrument to undertake these measurements is the differential scanning calorimeter (DSC). The heat input and temperature rise for the material under test are compared with those for a standard material both subjected to a controlled temperature programme. The general principles of DSC are outlined in a draft ISO standard, ISO 11357-1: 1997, for polymers and it contains a bibliography giving references to more detailed information.

Glass transition

It is particularly important to understand the effects that heat has on thermosetting polymers and on thermosetting polymer composites. The glass transition temperature (T_g) is the temperature below which the physical properties of a wholly or partially amorphous polymer varies in a manner similar to that of a solid phase (glassy state) and above which it behavies in a manner similar to that of a liquid (rubbery state). The T_g is the mid-point of a temperature range in which the wholly or partially amorphous polymer gradually becomes more viscous and changes from being a solid to a liquid state; this value is shown in **Figures 2(a)** and **2(b)**, as typical mechanical/physical properties of the polymer, namely the modulus of elasticity and hardness of the polymer plotted against temperature. The epoxies used in construction would generally be in the amorphous state, with a small amount of crystalline structure. The T_g of thermoplastic polymers (crystalline solids) are more complex than those of the thermosetting polymers because, in addition to a melting temperature T_m, **Figure 2(c)**, which takes place over a range of a few degrees and above which all their crystalline structure disappears, they also have a second but lower T_g below which they become rigid and brittle and can crack and shatter under stress. Most crystalline polymers possess some degree of an amorphous structure, thus they can have both a glass transition temperature (the amorphous portion) and a heat distortion temperature (the crystalline portion). The effect of crystallinity on properties is observed when the polymer is at a temperature between the T_g and the crystalline melting point. Above the glass transition temperature, the mobility of the polymer is great until the second thermal transition point is reached, namely the T_c, the crystallisation temperature of the polymer, at which point there is a fall in the heat flow and the molecules become more orderly. For 100% amorphous polymer this point does not appear. On further heating of the crystalline polymer the mobility of the molecules of the polymer again increases until the final thermal transition point, the melting temperature, T_m, is reached; the polymers will then melt. **Figure 2(c)** illustrates the typical heat flow and temperature of a crystalline and semi-crystalline polymer. From the results of the DSC technique it is possible to determine the degree of crystallinity of an amorphous/crystalline polymer. Thus all polymers below the T_g are rigid; they therefore have both stiffness and strength. Above the T_g, the amorphous polymers are soft

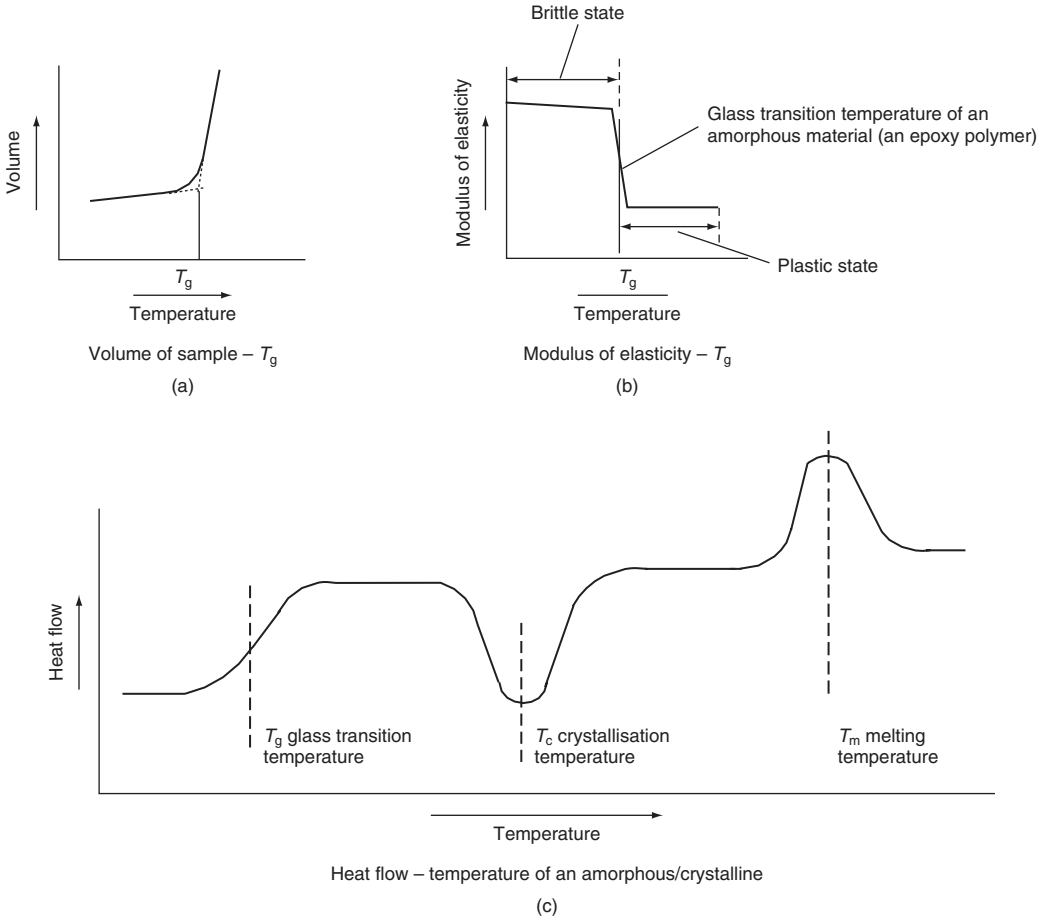

Figure 2 Modulus of elasticity, hardness and heat flow of an amorphous/crystalline polymer

elastomers or viscous liquids, consequently, they have no stiffness or strength, but the crystalline polymers will range in properties from a soft to a rigid solid depending upon the degree of crystallinity. The numerical value of the T_g may be quoted with slightly different values depending upon the testing technique used, these methods are the dynamic mechanical thermal analysis (DMTA) (ISO/CD 6721–11: 2001) and the differential scanning calorimetry (DSC) (ISO 11357-1: 1997).

The DMTA is a technique for monitoring the resultant strain by the excitation of a material with a periodic stress. This technique provides visco-elastic properties of a material with respect to temperature, humidity, frequency of vibrations-dynamic or static amplitude against time. ISO 2856 covers the dynamic testing and separates it into two broad classes: free vibration and forced vibration. The free vibration technique involving a torsion pendulum (covered in ISO 4663, 1986, Chapter 9 'Dynamic stress and strain properties') is limited to very low strains and frequencies. This method is infrequently used and currently the forced non-resonant test procedure is the one employed; this technique is covered in ISO 6721-5: 1996; ISO 2856: 1996, Chapter 5 'Conditioning and test atmospheres'; ISO 4664-1 'Rubber TC 45 test standard'; ASTM D2231, *Forced Vibration of Rubber*; and DIN 53513, *Dynamic Modulus of Elasticity and Dynamic Shear*.

Differential scanning calorimetry (DSC) is a thermal analytical technique in which the difference in the amount of heat required to increase the temperature of a sample and an inert reference material are measured as a function of temperature. Both the sample and reference are maintained at nearly the same temperature throughout the experiment. Generally, the temperature programme for a DSC analysis is designed such that the sample holder temperature increases linearly as a function of time. The reference sample should have a well-defined heat capacity over the range of temperatures to be scanned.

Hardness (volume–temperature is shown in Figure 2(a))

The term hardness, when referring to polymers, is a measure of resistance to indentation. The mode of deformation

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under an indentor is a mixture of tension, shear and compression; hardness is not a fundamental property. The result depends upon the indentor geometry and the degree of indentation as well as the time of indentation after which the measurement is made. Regardless of the arbitrary nature of the test, it is attractive because of its cheapness and apparent simplicity.

Modulus of elasticity (modulus of elasticity–temperature is shown in Figure 2(b))

From the above discussion it will be clear that polymer composite structural members should not be exposed to temperatures above the T_g value of the matrix material. Hot-cured thermosetting polymers would normally have a higher T_g value than the room temperature cured thermosetting materials; their actual values will generally be about 15–20°C above the curing temperature. The T_g of some low-temperature (ambient-cured) moulded composites can be increased in value by further post-curing the polymer at a higher temperature than that of the original cure but there is a maximum value of the T_g of all polymers irrespective of the post-cure temperature value.

Coefficient of thermal expansion (CTE)

By definition, the CTE is the change in length per unit length per unit rise in temperature; polymer material CTEs are of the order of 100×10^{-6} and therefore are an order higher than that of the more conventional civil engineering materials. This characteristic is an important consideration in structural design if metal inserts are to be placed in a moulded part or an adhesive polymer is used to join two dissimilar materials. It is important, particularly with the first case, to verify with temperature excursions that the maximum tensile strength of the polymer is not exceeded. The CTEs vary significantly with temperature ranges and they are calculated as the slope of the secant line of the thermal expansion curve between the reference temperature (generally the normal environmental temperature) and the temperature of interest. The rate of CTE of thermosetting polymers is influenced mainly by the degree of the cross-linking of the molecules of the material and the overall stiffness of the units between the cross-linkages.

Thermal conductivity

The thermal conductivity represents the rate at which heat is transferred by conduction through a given unit of the polymer when the temperature difference or gradient is normal to the surface area. The coefficient of thermal conductivity can be defined as the quantity of heat that travels through a unit volume of a polymer in a given time when the temperature gradient is 1 degree. Polymers have a low thermal conductivity compared with the more conventional civil engineering materials.

Chemical resistance

The resistance of the polymer to chemical attack, whether this is from the natural or a chemical environment, depends upon its chemical composition and the bonding in the monomer.

Permeability

Polymers with high crystallinity/density or a high degree of cross-linking will generally have low permeability, thus gasses and other small particles will not readily permeate through it. Haque *et al.* (2003), Liu *et al.* (2005), Hackman and Hollaway (2006) have shown that the ingress of moisture will permeate through polymers over time, particularly if the polymer (composite) is permanently immersed in water or salt solution, or is exposed to de-icing salt solutions.

In-service properties of the thermosetting polymers

As mentioned, two main functions of the polymer are to protect the fibres from hostile environments and bind and hold the fibres together in the required positions. The most important in-service properties of polymers are:

a *The strength of thermosetting polymers.* The bonding, length, density and degree of cross-linking of the molecular structure of the polymer is a function of the short-term strength of a thermosetting polymer. It will also depend upon the type of loading applied to that polymer. The long-term strength of the polymer will be dependent upon its durability in the environment into which it is placed. The durability of composites which is largely dependent upon the durability of the polymer. (Durability is discussed later in this section.)

b *The stiffness of thermosetting polymers.* The stiffness of the polymer is a function of its degree of cure, which in turn is a function of the degree of cross-linking of the 3D network of polymer chains. As with the strength of the polymer, its long-term stiffness will be dependent upon its durability in the site environment.

c *Thermal conductivity of all polymers.* The thermal conductivity of polymers is low and consequently, they are good heat insulators. By incorporating metallic fillers at the time of manufacture of the polymer can reduce its thermal conductivity.

d *Toughness of a thermosetting polymer.* The toughness of a polymer is a measure of the energy a sample can absorb before it fractures. It is estimated by calculating the area underneath the load–deformation curve (or stress–strain curve) from origin to the point of rupture of a tensile polymer sample; this area is shown hatched underneath the curve (ii) in **Figure 1**. It is important to

stress that because a material is strong it is not necessarily tough. In the figure, the curve (ii) has a lower modulus than that of curve (i), but has a greater deformation than curve (i); curve (ii) therefore has characteristics of toughness and strength, whereas curve (i) has strength but little toughness. The characteristics of curve (iii) have neither strength nor toughness.

An increase in thermal resistance and yield stress will lead to a decrease in toughness, impact and fatigue resistance. It is generally necessary to increase toughness without sacrificing thermal and mechanical properties. The toughening of thermosetting matrices has been achieved through various methods (Girard-Reydet *et al.*, 2002; Singh *et al.*, 2002). There are two methods for improving the toughness of thermosetting polymers and these are similar to those used for amorphous or semi-crystalline thermoplastic polymers, viz. by plasticisation or by the amplification of the deformation mechanism. In the plasticisation process, a miscible low-T_g compound is added to the formulation, so as to produce a decrease in both the glass transition temperature and the yield stress but a corresponding improvement in its fracture resistance. However, there are drawbacks to this method and, consequently, it is generally not used for toughening purposes. The more successful method for the improvement in the toughness of thermosetting polymers is by the addition of a suitable elastomer to the uncured resin and the control of the polymerisation reactions in order to induce phase separation (Huang and Kinloch, 1992; Chikhi *et al.*, 2002). However, when using this method it is necessary to limit the elastomer content because at concentrations higher than 15–20 wt% the elastomer reduces both the elastic modulus and thermal performance of the mixture. Another method investigated in recent years is the modification of these resins with core-shell particles or high-performance thermoplastic polymers with their high modulus and glass transition temperatures. When considering these temperatures it is seen that an increase in the toughness of the polymer will tend to decrease its stiffness; consequently, a compromise between strength and stiffness of a thermosetting polymer must be made. The principles of toughening of thermosetting polymers have been described by Birch and Williams (1978), Riew and Kinloch (1999) and Hans-Henning (2005).

Dimensional stability

All thermosetting polymers depend upon intermolecular cross-links for their strength and as the temperature increases the polymer tends to rise to its (T_g) value and begins to soften. The temperature at which this happens depends upon the detailed chemical structure of the polymer. The hot-cured polyesters, vinyl-esters and the epoxies all begin to weaken and break down at above 200°C; clearly, site polymers should not be exposed to temperatures above their T_g value. The automated manufacture of hot-cured thermosetting polymers would have a higher T_g value than the ambient temperature (cold)-cured ones; the hot-cured polymer would, as stated above, generally be about 10–20°C above the curing temperatures (between about 120–150°C). The T_g of *some* low temperature cured composites can be increased in value by further post-curing the polymer to a higher temperature than its initial cure but there is a maximum value of post-cure temperature beyond which the T_g, cannot be raised. Furthermore, polymers with a high crystallinity will have a region of acceptable dimensional stability above T_g. At the *thermodynamic melting point* (T_m) crystalline polymers immediately change into liquids and by so doing will have zero mechanical properties.

Durability

The aging of a polymer can be defined as a slow and irreversible variation of a polymer's material structure, morphology and/or composition, leading to a deleterious change of use. The causes of this change can be the instability of the material during in-service use or its interaction with the environment. It is a difficult problem to study because it proceeds very slowly (typical lifetime in years). Accelerated testing can be undertaken to build kinetic models that describe the time changes of the material's behaviour and then to use these models to predict the durability from a conventional lifetime criterion. It is then necessary to show the pertinence of the choice of accelerated aging conditions, and the mathematical form of the kinetic model and the lifetime criterion have to be proved. Empirical models are highly questionable because they have to be used in extrapolations for which they are not appropriate. Hollaway (2007) has discussed some field surveys which have been undertaken on FRP composite materials throughout the world and the necessity to treat with caution/care the results of accelerated experimental testing, particularly if the polymer is heated during the testing.

The mechanical properties of the thermosetting polymers

The polymer composite derives its mechanical characteristics wholly from those of the fibre, therefore the most important property of the polymer is its in-service characteristics, which have been discussed in this chapter. However, it is important for the civil engineer to understand the mechanical properties of the polymer. **Table 1** gives typical values of the modulus of elasticity for different thermosetting polymers.

Ultimate tensile strength

A thermosetting polymer exhibits different forms of strength characteristics depending upon the loading to

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Material	Specific strength	Ultimate tensile strength: MPa	Modulus of elasticity in tension: GPa	Coefficient of linear expansion: 10^{-6}/°C
Thermosetting				
Polyester	1.28	45–90	2.5–4.0	100–110
Vinylester	1.07	90	4.0	80
Epoxy	1.03	90–110	3.5	45–65

Table 1 Typical tensile mechanical properties of the three thermosetting polymers used in civil engineering

which it is subjected, for example when it is exposed to static short- or long-term loading or dynamic or impact loading. The short-term 'yield' strength of a polymer is dependent upon the bonding between the molecules; generally, the higher the degree of cross-linking the higher will be the 'yield' strength. If substantial heating is encountered during the fatiguing of the material, all factors that influence thermal stability will also influence fatigue strengths. Typical tensile mechanical properties of the main thermosetting polymers used in civil engineering are given in **Table 1**.

Poisson's ratio

The Poisson's ratio is the ratio of the contraction to the elongation in the loading direction and will be dependent upon loading conditions.

Creep characteristics of polymers

The deformation of a polymer material over time under the application of a load is referred to as the creep of that material; this deformation will continue as long as the load is applied. Upon removing the load the polymer will regain some of its original length but will not return completely to its original condition. The reason for this characteristic is that a polymer has both the properties of an elastic solid and a viscous fluid, thus it is classified as a visco-elastic material; it is the viscous component that does not allow it to return to its original condition. The creep, flow and plastic deformation in the polymeric materials result from the irreversible slippage, decoupling and disentanglements of the polymer chains in semi-crystalline polymers. The creep of the polymeric material depends upon:

1. the time-dependent nature of the micro-damage in the composite material subjected to stress;
2. molecular characteristics and microstructure;
3. the loading histories and the nature of the applied load;
4. the temperature and moisture environments into which it is placed.

In assessing the creep of a polymer material it is particularly important to ensure that the service temperature does not approach the glass transition temperature of the polymer.

Figure 3 demonstrates a family of creep curves consisting of an isostrain creep curve (**Figure 3(a)**), an isostress creep curve (**Figure 3(b)**) and an 100 s isochronous stress–strain creep curve (**Figure 3(c)**). The first and third curves have been produced by cross-plotting, from the isostress creep curves at constant times.

BS 4618-5.3: 1972 specification requires that constant load tests are carried out under controlled conditions for the following durations 60 s, 100 s, 1 h, 2 h 100 h, 1 year, 10 years and 80 years; isochronous stress–strain curves, relevant to a particular time of loading, can then be produced. By varying the stress a family of creep curves may be obtained as shown in Figure 2.3c; isochronous stress–strain curves will correspond to a specific loading direction. The 100 s isochronous stress–strain curve is a plot of the total strain (at the end of 100 s) against the corresponding stress level. The creep modulus of the material may then be found by measuring the slope at any specific stress level on the isochronous stress–strain curve; the creep modulus will vary for every stress level.

Alklonis and MacKnight (1983) developed the time temperature superposition principle (TTSP). The principle is a sophisticated approach to obtain creep characteristics of polymers through master curves for creep, creep compliance or stress compliance against time (or *log* time). When applied to the generation of a creep master curve the following steps are required.

- A master specimen is subjected to a constant load at a certain temperature as in conventional creep tests.
- Similar experiments are performed for identical specimens at different temperature levels and the relevant creep curves drawn.
- An arbitrary reference temperature is selected.
- All the individual creep curves corresponding to different temperature levels are shifted along the (*log* time) coordinate to obtain the superimposed master curve.

The shift process is shown diagrammatically in **Figure 4**.

Cheng and Yang (2005) developed the (TTSP) further by introducing a matched theoretical calculated curve from a supposed model of transition kinetics in which only time is involved as the independent variable. Cessna (1971) developed the time applied stress superposition principle (TSSP); this principle can also estimate creep values for polymers and polymer composite materials.

The fibre

A wide range of amorphous and crystalline materials can be used to form fibres, but in the construction industry the three fibres that are generally used in structural systems are the glass fibre, the aramid fibre and the carbon fibre.

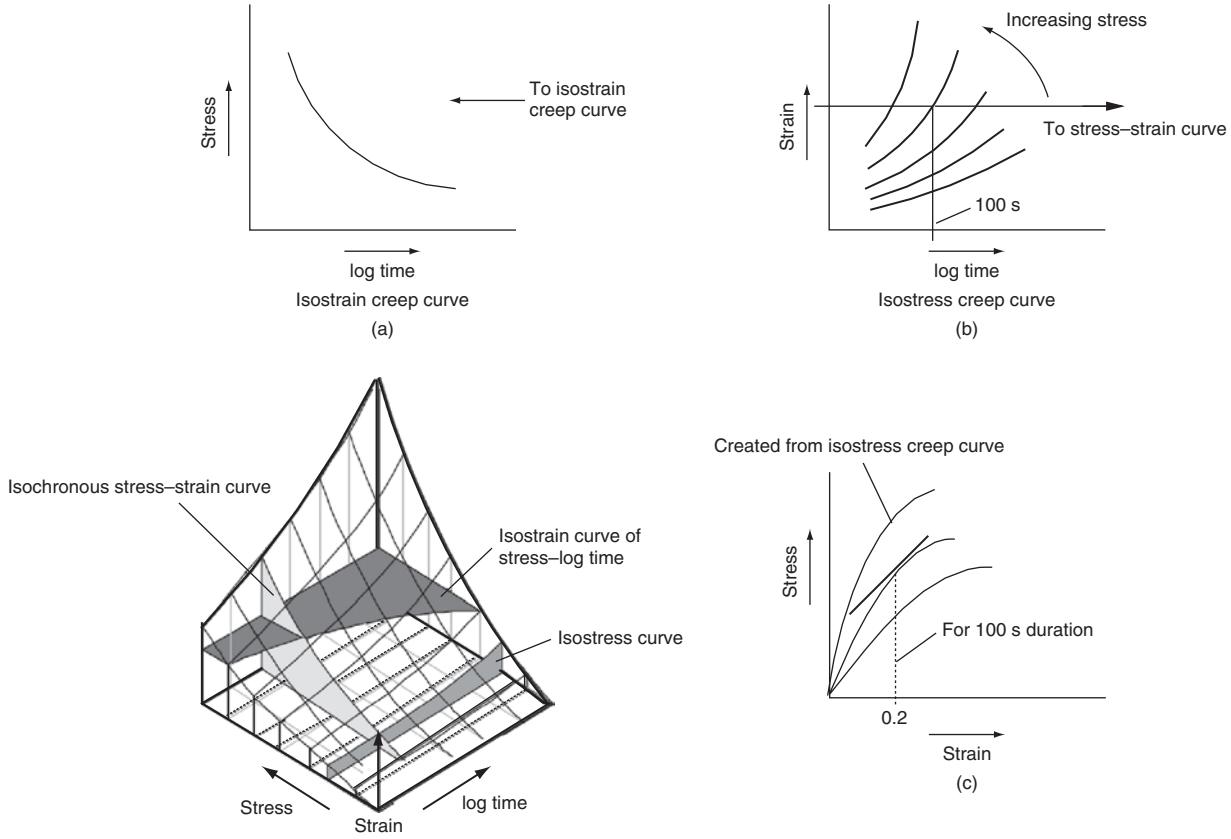

Figure 3 Creep curves for a thermosetting polymer

The fibres may be used separately or as a hybrid of two or three different fibres. The hybrid system will be used when, say, the major fibre is the glass fibre but the stiffness is required to be increased somewhat and this could be achieved by introducing some carbon fibre into the system.

Assembled rovings are used in processes involving discontinuous and continuous fibres and sheets. These techniques have been discussed in Chapter 52 and in Phillips (1989) and Hollaway (1993). The simultaneously drawn filaments are used in the fabrication of several types of yarns; rovings may be used either individually or in the form of a woven fabric.

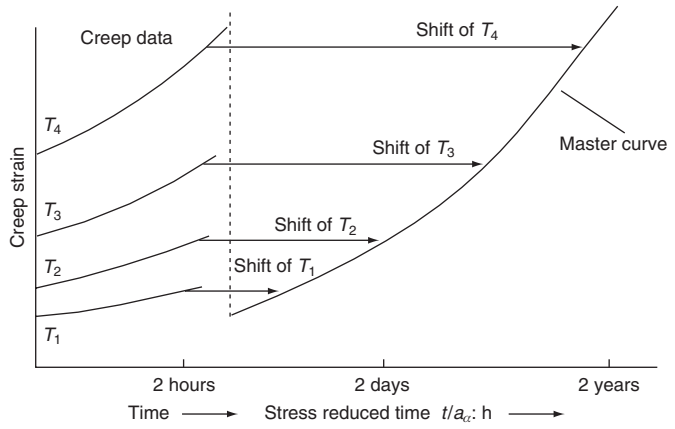

Figure 4 Stress time superposition principle (TSSP) for thermosetting polymer

The main synthetic fibres used in civil engineering are the polypropylene, the polyethylene, the polyester and the polyamide, and to provide the required engineering applications these fibres are able to be engineered chemically, physically and mechanically. Natural fibres such as cotton and jute and the regenerated cellulose and rayon fibres are not generally used because they are biodegradable. However, they could be used as geotextiles which are required for temporary functions and where biodegradation is desirable; for example, a practical case where it is necessary for temporary erosion control. The mechanical properties of civil engineering synthetic fibres are given in **Table 2**.

The glass fibre

The basis of the glass fibre is SiO_2, In its pure form it exists as $(SiO_2)_n$ and has no melting point as such but softens at about 2000°C; at this temperature it commences to melt. Pure silica is a viable glass fibre but it must be worked at

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Fibre type	Ultimate tensile strength: MPa	Specific weight	Maximum working temperature: °C	Elongation at break: %
Synthetic				
Acrylics	300–400	1.14–1.18	180	20–40
Modacrylics	300–400	1.31–1.37	170	20–40
Fluorofibres	105–175	2.30	220	40–100
Chlorofibres				
Polyvinyl chloride	340	1.38–1.40	80	20
Chloride	300–400	1.70	80	20–40
Aramid	500–2900	1.38–1.45	450	2–12
Nylons				
Nylon 6	450–750	1.13	70	20–40
Nylon 66	450–1000	1.14	130	12–30
Polyolefins				
Polypropylene	400–700	0.9	100	10–20
Polyethylene	80–120	0.92	55	25–50
low density	350–500	0.95	65	20–30
high density	2400–2700	0.96	90	3–4
ultra high density				
Polyurethane Elastomers	70–110	1.00–1.32	150	400–800
Polyesters	500–1400	1.38	170	8–30
Vinyls	200–700	1.3	140	10–15

Table 2 Typical mechanical properties of synthetic fibres

a very high temperature, which is a disadvantage unless its specific chemical properties are required. For civil engineering applications impurities are added to the glass in the form of other metals to lower its working temperature. These materials will impart on other properties of the glass which may be beneficial for different applications.

The first type of glass used for the manufacture of glass fibre was:

1. Soda-lime glass, known as A-glass but it is not resistant to alkali.
2. E-glass, which is the type generally used in construction, it is an alumino-borosilicate glass, it has a low alkali content of the order of 2%. It is named E-glass because it was originally used for electrical applications. This glass it is not resistant to alkali.
3. S-glass, compared with E-glass, is stronger (typically 40% greater at room temperature), and stiffer with a greater corrosion resistance. It has good heat resistance and resistance to acids such as hydrochloric, nitric and sulphuric acids.
4. C-glass has been developed to resist attack from chemicals, in particular acids.
5. T-glass is a North American variation of C-glass.
6. AR-glass is an alkali-resistant glass, specifically used for resisting alkali in Portland cement and is therefore used as the reinforcement for cements glass fibre reinforced composite (GFRC). It is used in pultruded rebars to reinforce concrete, although the E-glass fibre is the one generally used in this manufacturing system.

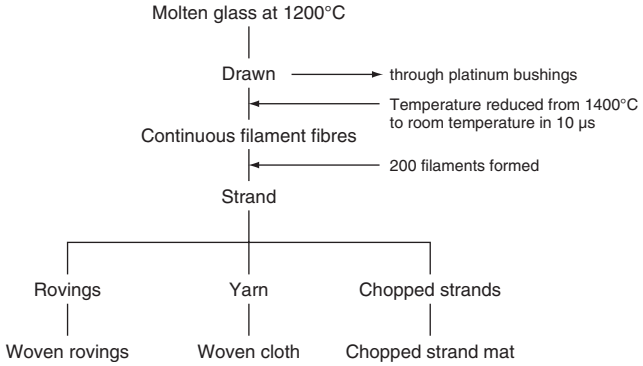

Figure 5 Diagrammatic procedure for the manufacture of glass fibre

The glass fibre is made initially from a marble remelt process. The raw material in solid form is melted in a furnace; it is sheared and rolled into marbles which are cooled and packaged. The marbles are then received by the manufacturing facility where they are remelted and the molten glass at 1200–1400°C is drawn through platinum bushing to be made into fibres. Platinum alloyed with rhodium for durability is used as the glass melt has a natural affinity for wetting. The number of nozzles, each forming a filament of fibre, ranges from 200 to 4000 in multiples of 200; each 200 nozzles produce 200 individual filaments or one fibre, and fibres are then bundled together to form strands. The filaments have exceptionally high specific stiffness and strength and range in diameter from 3 to 24 μm. A surface treatment or sizing is applied before the fibres are gathered into strands and wound onto a drum; the size protects the fibre during winding. **Figure 5** gives a line diagram of the manufacturing procedure. Some sizes, gelatinous mixtures, are used to make the fibres have an affinity for certain resins, if the fibre is to be used in a composite.

The aramid fibre

Kevlar 49 is a member of the group of highly crystalline aramid (aromatic polyamide) fibres that have a low specific weight, and the highest specific tensile strength and the lowest specific weight among the current reinforcing fibres. The molecular structure of Kevlar 49 is a repeating unit in its molecules containing an amide and an aromatic ring. The latter gives the molecules higher chain stiffness as well as a better chemical and thermal stability over other commercial organic fibres. There are a number of companies worldwide manufacturing aramid fibres but the most common materials in the USA and Europe are manufactured by Du Pont as Nomex and Kevlar; the

latter is the brand name for a light but strong synthetic fibre. There are three common types of Kevlar, namely Kevlar, Kevlar 29 and Kevlar 49; it is the Kevlar 49 that is used in construction.

Kevlar 49 filaments are manufactured by the condensation of terephthaloyl chloride and *p*-phenylene diamine from a spinneret. During the drawing of the filament, the molecules of Kevlar 49 become highly oriented in the direction of the filament axis. In the transverse direction the weak hydrogen bonds between the hydrogen and oxygen atoms in adjacent molecules hold them together. The resulting filament is highly anisotropic with longitudinal direction having the better physical and mechanical properties.

Kevlar 49 fibres do not melt or support combustion but will commence to carbonise at about 427°C; the fibre is heat and cut resistant. The maximum long-term use of the fibre exposed to a temperature is 160°C and the thermal conductivity of the fibre is very low. The chemical resistance of the fibre is good with the exception of a few strong acids and alkalis; the fibre is sensitive to ultraviolet light. Prolonged exposure to direct sunlight causes discolouration and significant loss in tensile strength. The fibre is hygroscopic and can absorb up to 6% of moisture at 100% relative humidity.

Kevlar 49 possesses good toughness and damage tolerance properties. It is resistant to fatigue, both static and dynamic, and is elastic in tension but behaves non-linearly in compression and, in addition, has a ductile compressive characteristic. Its strength is not affected by immersion in water and it does not rust or corrode. The main weakness of the aramid fibre is that it decomposes under alkaline and chlorine environments.

Kevlar is expensive, partly because of the difficulties caused by the use of concentrated sulphuric acid in its manufacture. The harsh conditions are required to keep the insoluble polymer in solution during synthesis and spinning. It is supplied in reinforcement form as rovings or as yarn for weaving into fabric. Yarn is available from about 20 tex to 300 tex, and rovings are around 500 tex to 1000 tex.

The second generation of Kevlar fibre is the Kevlar 149 which has the highest tensile modulus (40% higher than Kevlar 49) of all commercially available aramid fibres; however, the strain to failure is low. The equilibrium moisture content (1.2% at 65% relative humidity) is nearly 70% lower than that of Kevlar 49, it also has a lower creep rate than that of Kevlar 49.

The carbon fibre

Carbon fibres are manufactured by controlled pyrolysis and cyclisation of certain organic fibre precursors (the raw material used to make carbon fibre). About 90% of carbon fibres are produced from polyacrylonitrile (PAN); the remaining 10% are made from rayon or petroleum pitch. The PAN precursor is manufactured by spinning to produce a round cross-section fibre; the yield is only 50% of the original precursor fibre. The PAN precursor carbon fibres can also be manufactured by a melt-assisted extrusion as part of the spinning operation. I-type and X-type rectangular cross-section carbon fibre composites are produced with a closer fibre packing in the composite compared to that of the circular fibre. The PITCH precursor fibres are derived from petroleum, asphalt, coal tar and PVC; the carbon yield is high but the uniformity of the fibre cross-sections is not constant from batch to batch. This non-uniformity is acceptable to the construction industry; in addition, the pitch fibre precursor is invariably used when the ultra-high stiffness carbon fibres are required.

These materials are organic polymers, characterised by long strings of molecules bound together by carbon atoms. The exact composition of each precursor varies from one manufacturing technique to another. Carbon fibres are classified according to their mechanical properties as high modulus high strength and ultra-high modulus fibres. Carbon filaments are typically between 5 μm and 8 μm in diameter and are combined into tows containing between 3000 and 12 000 filaments. A tow count is typically between 200 tex and 900 tex. The tows are twisted into yarns and woven into fabrics analogous to those described for glass.

For the construction industry most commonly, carbon fibres are produced from the polymer PAN, consequently only the manufacturing of this type of fibre will be considered. The basic techniques for the high modulus (H-M) and the ultra-high modulus (UH-M), European definitions of the fibres, are the same but the heat treatment temperature will be greater the higher the modulus of the fibres, thus, a more highly orientated fibre of crystallites will be formed for the UH-M fibre. A typical sequence of operation used to form carbon fibres from polyacrylonitrile (PAN) include: (i) spinning; (ii) stabilisation; (iii) carbonisation; (iv) graphitisation; (v) surface treatment; and, finally, (vi) sizing of the fibre.

1. *Spinning*: acrylonitrile plastic powder is mixed with methyl acrylate or methyl methacrylate and is reacted with a catalyst in a solution polymerisation process to form a polyacrylonitrile plastic. This plastic is then spun into fibres, this process is important as during the spinning the internal atomic structure of the fibre is formed. The fibres are then stretched to the required diameter and during this operation the fibre molecules are orientated into one direction along the longitudinal direction of the fibre.
2. *Stabilisation*: the fibres are heated in air to about 200–300°C to convert their linear atomic bonding to a thermally stable bond. During this process the fibres combine with oxygen molecules in the air and rearrange

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their atomic bonding pattern, this process involves techniques of applying heat to the fibres and mixing them with certain gasses.

3 *Carbonation*: the stabilised fibres are heated to a temperature of between 1000°C and 3000°C for several minutes in a furnace with a gas that does not contain oxygen, this prevents the fibre from burning in the high temperature. During this process the fibres lose their non-carbon atoms. The carbon atoms form tightly bonded crystals that are aligned to the longitudinal axes of the fibre.

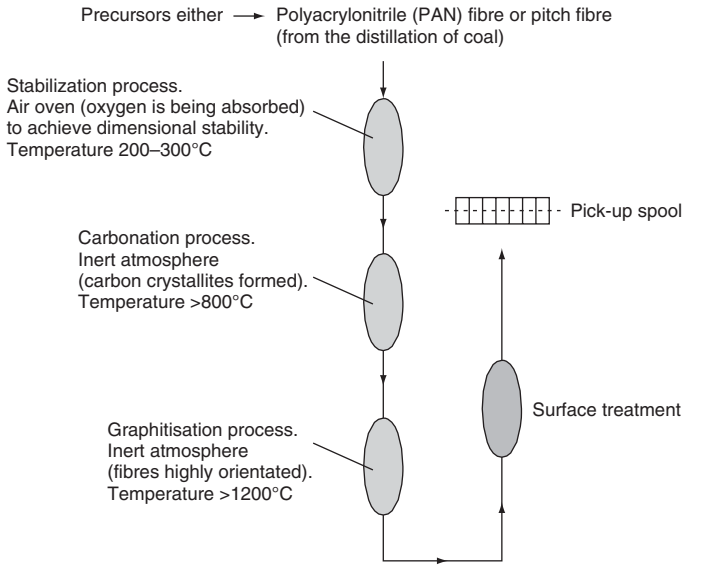
4 *Graphitisation*: involves heating the polymer in excess of 2500°C in a nitrogen-rich environment, which expels impurities until the polymer contains 92–100% carbon, depending on the quality required for the fibre. At the same time the fibres are stretched and this results in their sliding over each other and a further orientation parallel to the fibre axis.

5 *Surface treatment*: the surface of the carbon fibre does not bond very well to epoxies and consequently their surfaces are slightly oxidised. This procedure provides a better chemical bond and as the surfaces are etched and roughened through oxidisation it provides mechanical bond also. The process must be carefully controlled to prevent surface defects in the fibre which could cause premature fibre failure.

6 *Sizing of fibre*: to protect the surface of the fibre from damage during winding or weaving it is coated with a material which must be compatible with the adhesive material used to form the composite.

A schematic representation of the production of carbon fibre is given in **Figure 6**.

In addition to strong and high stiffness fibres, civil engineering requires in-service resistance to withstand high temperature and aggressive environmental conditions; carbon fibres, in general, are not affected by moisture, solvents, bases and weak acids.



Polyacrylonitrile fibres for production of high-modulus fibres (construction industry) or production of high-modulus or ultra-high modulus (aerospace industry)
Pitch fibres for production of ultra-high modulus carbon fibres (construction industry)
(Definitions of fibres used here are the European ones)

Figure 6 Diagrammatic representation of the manufacture of carbon fibre

Mechanical properties of fibres

Tensile strength and stiffness

The tensile strength is one of the most important physical characteristics of the reinforcing fibre; accurately quantifying strength and variability in strength is of great importance. The fibre strength measurements are used by the fibre and the composite developers and by the manufacturers as a means of predicting composite properties as assessing the consistency of different groups of fibres. The testing of the fibres is covered in the chapter on failure criteria of composites.

The variability of strength within a sample of fibres is important. Models have indicated the strength of composites is determined not only by the mean tensile strength of the reinforcing fibre but also by the distribution of strength of the fibres. This occurs as the failure of a few weak fibres within the composite at low loads can lead to failure of the entire composite. Weibull statistics are commonly used to predict strength at different tested volumes (gauge length, area), to predict strength of bundles of fibres and ultimately to predict the strength of fibre-reinforced composite.

Fibres used for the reinforcement of polymers behave as brittle elastic solids and therefore exhibit a linear stress–strain relationship; consequently, the strength of the fibre is defined as the load at failure divided by the cross-sectional area of the fibre.

The tensile mechanical properties of the glass, aramid and carbon fibres are given in **Table 3**.

In-service properties

Creep

The creep characteristics of glass, aramid and carbon fibres are very small and are not generally considered in the design of polymer composite components for civil engineering.

Durability

E-glass fibres are most commonly used in GFRP applications, due particularly to their excellent tensile strength and low cost when compared with other fibres. However, all glass fibres are very susceptible to alkaline environments, which is not the case with carbon or aramid fibres; their

Material	Fibre	Elastic modulus: GPa	Tensile strength: MPa	Ultimate strain: %
Glass fibre	E	69	2400	3.5
	A	69	3700	5.4
	S-2	86	3450	4.0
Carbon fibre	HM[1]	300	5200	1.73
Pan-based fibre	UHM[2]	450	3500	0.78
Hysol Grafil Apollo	HS[3]	260	5020	1.93
Pan-based fibre	G-40-700	300	4960	1.66
BASF Celion	Gy 80	572	1860	0.33
Pan-based fibre	T-300	234	3530	1.51
Torayca				
Pitch-based fibres	T-300	227.5	2758.0	1.76
Hysol Union carbide	T-500	241.3	3447.5	1.79
	T-600	241.3	4137.0	1.80
	T-700	248.2	4550.7	1.81
Aramid fibre	49	125	2760	2.2
	29	83	2750	3.3

Notes:
1 High modulus (European definition)
2 Ultra high modulus (European definition)
3 High strain

Table 3 Typical tensile mechanical properties of glass, carbon and aramid fibres

susceptibility is primarily due to the presence of silica in the glass fibres. The silica reacts with hydroxyl ions in the surrounding alkaline environment based on the following chemical reaction:

$$SiO_2 + 4OH^- \rightarrow SiO_4^4 + 2H_2O$$

After this initial reaction, the SiO_4^4 ions continue to react with metal ions (i.e. Ca^{2+}, K^+, and Na^+) in the alkaline solution to produce hydrated silicates (i.e. calcium silicate hydrate) on the surface of the fibres (Holmes *et al.*, 1991). As the reaction progresses, the glass fibres lose stiffness and tensile strength as the silica is consumed. The resultant effect is an overall loss of composite strength.

Laboratory tests to estimate the long-term durability performance of glass fibres encapsulated in a polymer matrix, resulting in the above equation, would be defined as an accelerated test; these types of tests require considerable understanding. Many such studies have been performed and conclusions drawn which have completely misrepresented the situation. An example has been mentioned in Hollaway (2007) concerning the ingress of salt solutions into GFRP composite rebars which, from accelerated tests using concentrated salt solutions, have indicated a serious alkali attack on the glass fibre; in practice, however, it was shown that no such attack took place.

Glass fibres are now on the market that are more resistant to alkaline environment and can be used to increase the durability of the composite. Advantex is a glass fibre manufactured by Ownes Corning and when it is embedded into a polymer matrix and immersed into a simulated concrete pore water solution and loaded to 30% of its ultimate load capacity it can, as reported by Benmokrane *et al.* (2002), retain 100% of its strength after 140 days of immersion. Conventional E-glass fibres exposed to the same conditions resulted in 16% loss of tensile strength. To reduce the diffusion rates of moisture and alkali ions, Benmokrane *et al.* (2002) showed that vinylester resin matrix provides a better protection for GFRP bars than polyester resin matrix and by applying sand coatings to the FRP bar reduced the micro-cracks and macro-cracks in the resin coating surface; this occurs due to chemical reaction between the coating resins and alkali ions. Saint-Gobain Vetrotex produce a similar alkaline-resistant glass, known as ARcoteXTM, which has been proven to increase the durability of glass composites (Almenara and Thornburrow, 2004). Parameters which influence the diffusion of moisture and alkali ions through the FRP bar, and as a consequence its durability, are fibre content, manufacturing process, exposure time and temperature. The latter must not exceed the glass transition temperature of the matrix or the temperature at which different mechanisms of degradation may creep; this temperature has been suggested as 60°C.

Carbon fibres do not absorb liquids and are subsequently resistant to all forms of ingress from alkalis or solvents (Ceroni *et al.*, 2006).

Aramid fibres have been reported to suffer some reduction in tensile strength when exposed to an alkaline environment (Balazs and Borosnyoi, 2001), however, the long-term properties of a matrix for CFRP and AFRP composites are still of importance to the overall properties of the composite; the matrix protects the fibre from external influences.

Effects of hydrolysis

Most glass fibres have limited solubility in water but they are very dependent upon the pH value of the liquid. Chloride ions will also attack and dissolve the surface of the E-glass fibre. Moisture is readily adsorbed and can exacerbate microscopic cracks and surface defects in the fibre and thus reduce the tensile strength of the glass fibre. The glass fibres have high surface area to weight but the increased surface makes them much more susceptible to chemical attack. **Table 4** shows the chemical composition of E-glass and S-glass.

Oxides used	CaO	Al_2O_3	MgO	B_2O_3	SiO_2
For E-glass	17.5	14.0	4.5	10	54
For S-glass	–	25	10	–	65

Table 4 The chemical composition of E-glass and S-glass (adapted from Phillips, 1989)

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Chapter 52

Advanced polymer composites

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The design and analysis of fibre-reinforced polymer composite structural members requires a detailed knowledge of the properties of the constituent materials which form the structural members; these have been given in the chapter on Characterisation of fibre and matrix materials used in construction. The manufacture of advanced polymer composites is by the impregnation of the matrix material into the fibre. The mechanical properties of the composite will be highly dependent upon the method of manufacture, the arrangement, the direction and the volume fraction of the fibre; the fibre array in the polymer will dictate whether the material has the properties of anisotropy or isotropy. These points will be discussed in this chapter. The environment into which the composite is placed is important to enable the design of the in-service properties of it to be undertaken; the understanding of the long-term durability of the fibre–matrix composite is also of great importance and will be discussed in this chapter.

Introduction

The mechanical and in-service properties are important aspects of polymer composite materials and it will be shown that these are dependent upon various factors including the basic materials used, the methods available to impregnate the fibre with the polymer, and the interaction between the surface of the fibre and polymer. This chapter will concentrate upon the advantages of and the drawbacks associated with advanced polymer composites for use in a civil engineering environment and thence the benefits of these materials over the other more conventional construction materials.

The properties of composite material

There are seven main factors that govern the mechanical and physical properties of the composite, these are:

- The basic mechanical properties of the polymer and fibre are given in Chapter 51.
- The surface interaction of fibre and resin (the 'interface').
- The relative proportions of the polymer and fibre (fibre volume fraction).
- The orientation of the fibre (i.e. unidirectional (anisotropic composite), bi-directional (orthotropic composite) or randomly orientated (quasi-isotropic composite)).
- The method of manufacture of the composite.
- The creep of the composite.
- The long-term durability of the composite system.

The surface interaction between fibre and polymer

The final performance of a composite material depends strongly on the quality of the fibre–matrix interface. This interface is an anisotropic transition region that provides adequate chemical and physical bonding between the fibre and the polymer. In analysing fibre-reinforced matrix composite material, the primary aim is to obtain predictions of the average behaviour of the composite from the properties of the components, but these must act compositely for the material to be efficient. It is well known that the application of a coupling agent to, say, a glass-fibre surface will improve fibre–matrix adhesion in composites. However, on a commercial glass fibre the coupling agent forms only a small fraction of the coating, the larger part being a mixture of processing additives whose contribution to composite properties is not well defined. The interfacial region of the composite will therefore be affected not only by the composition of the coating but also by its distribution on the glass fibre surface and in the composite matrix.

The orientation of the fibre

The strength and stiffness of the composite will be determined by the direction of the fibres, i.e. unidirectional, bi-directional, angle ply or randomly orientated fibre. The relationship between the stiffness of the composite under a tensile load for different types of fibre arrays and the angle β of the fibre to the major axes (0° direction) is shown in **Figure 1**. It will be observed that for the unidirectional fibre composite the major axes direction (0° direction) of the fibre gives the greatest stiffness and the 90° axes direction of the fibre gives the least stiffness, this latter being equivalent to a value slightly less than that of the matrix stiffness, due to an 'impurity' (i.e. the fibre) in the matrix material. The randomly orientated fibre composite gives equal properties in all directions from 0° to 90° axes direction. Relatively little is known of the response to impact loading, which usually occurs at strain rates higher than those used to measure the quasi-static mechanical properties of materials. The tests

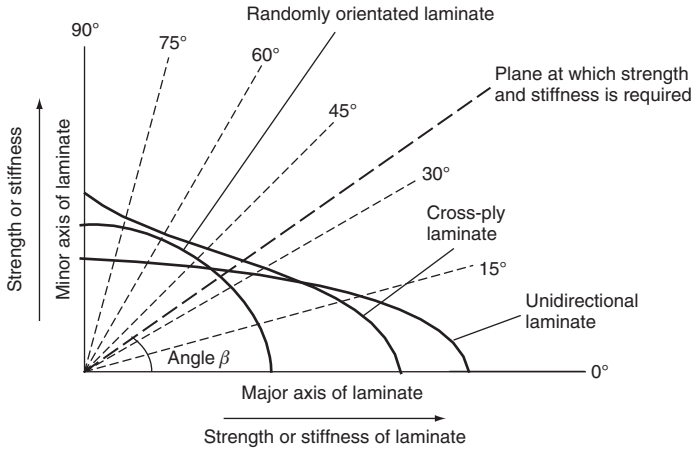

Figure 1 Relationship between stiffness/strength and plane at angle β

undertaken by Vinson (2001) show that changing the fibre orientation changes the values of the ultimate strength and strain of carbon–epoxy composites. The results also show that the ultimate strength generally decreases as the off-axis angle is increased; however, the decrease is not uniform as is the case for the quasi-static tests.

The method of manufacture of the composite

There are a number of techniques available for the manufacture of advance polymer composites and each process has an effect upon the mechanical properties of the final composite. The automated fabrication methods for the production of composites have a high degree of production control, composite compaction and curing compared to the manually fabricated techniques and therefore the former technique will have higher values of strength and stiffness compared to those of the latter methods.

The various manufacturing processes given here are all relevant to civil engineering and can be divided into three major divisions: (i) manual techniques; (ii) the semi-automated process; and (iii) the automated process. A short discussion of these processes and an indication when specific applications are used in practice is now given.

Manual techniques

These methods have been discussed in Hollaway (2008a) and their descriptions mentioned here have largely been taken from that publication.

The manual processes used currently by the construction industry are a variation of the general wet lay-up method. The commercial companies that manufacture advanced polymer composites (APCs) by this process are:

- the REPLARK method;
- the Dupont method;
- the Tonen Forca method.

These techniques are basically the same with minor variations. In the wet lay-up method the dry fibres in the form of sheets or fabrics are impregnated in situ with a polymer. The sheets or fabrics for strengthening structural members are wrapped around the member or placed on a mould and their size will depend upon the size of the member or mould; the fibre reinforcements are generally of widths varying between 150 mm and 1500 mm.

The Replark method

Developed by the Mitsubishi Chemical Corporation, it is a prepreg (pre-impregnated) carbon-fibre sheet where the matrix material is an epoxy resin (Epotherm) and the fibres are Mitsubishi-manufactured fibres, unidirectionally oriented. The sheet has a paper backing, which serves to keep the fibres in position; the backing is removed when the sheet is placed in position on the mould or on to the structure. The procedure employed to apply the carbon-fibre-reinforced polymers (CFRP) sheet is as follows.

- The first layer of the impregnating polymer is applied to the prepared surface of the mould or structural member, this layer acts as an adhesive as well as the matrix of the composite in rehabilitation of a structural member.
- The CFRP sheet cut to size is adhered to the concrete using the first laminating layer of polymer, it is pressed down on the surface using a roller to expel entrapped air between the fibres and the polymer.
- After the paper backing is removed from the carbon-fibre sheet the second layer of impregnating polymer is applied.
- The last two steps, application of polymer and sheets, are repeated in the case of multiple plies.
- The system is then allowed to cure and, if specified, a post-cure procedure is undertaken.

The Dupont method

This is a system using Kevlar fibres, which is marketed as a repair system for concrete structures. The application of the material to the surface to be rehabilitated/retrofitted is similar to the Replark method.

The Tonen Forca method

This is a unidirectional carbon-fibre sheet in an epoxy laminated system marketed in the UK by Kyokuto Boeki Kaisha Ltd. The system was originally developed by Mitsubishi Chemical Corporation and is therefore similar to the Replark system.

These methods have been used in bridge engineering to confine columns or for strengthening/stiffening bridge beams.

Further details on these methods may be obtained from Hollaway and Head (2001), Hollaway and Teng (2006).

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The semi-automated process

The semi-automated process used currently is:

- the hot-melt factory-made pre-impregnated fibre (prepreg).

The hot-melt factory-made pre-impregnated fibre (prepreg)

The hot-melt factory-made prepreg is factory cured, for the production of precast plates or sections, or site cured if the prepreg composite is fabricated on a structural member. In the latter case, a compatible film adhesive is used and the adhesive and the prepreg components are cured in one operation under an elevated temperature of 65°C applied for 16 h or 80°C applied for 4 h; a vacuum-assisted pressure of 1 bar is applied for simultaneous compaction of the composite and the film adhesive, Hollaway *et al.* (2006). It is forecast that this method will be used increasingly for strengthening/stiffening degraded buildings/bridges due to its superior fabrication process.

In the UK the manufacturing specialist in the production of hot-melt factory-made prepreg fibre for the construction industry is ACG at Derbyshire.

The automated processes

The processes that are available to the construction industry are:

- the pultrusion technique;
- the filament winding technique;
- the resin transfer moulding process.

The pultrusion technique

This technique is used quite extensively in the construction industry and a large percentage of this use is associated with the construction of 'all-composite' structures or the strengthening of bridges and structures. Sections are manufactured in a factory using a hot polymerised resin; the dies operate at temperatures between 120°C and 135°C. The sections produced will generally be fully cured but this does depend upon their size; the large sizes of the sections may require post-curing. Flat plates and various geometrical cross-sections can be produced; they are generally straight in the longitudinal direction although products can be manufactured which are curved in plan. Care must be taken to ensure that: (i) the fibres are well compacted into any bends in the cross-section (thus preventing voids forming); (ii) there is complete wetting of the fibres in the pultruded unit (again, preventing voids forming): wetting of the fibres takes the form of passing the fibres through a resin bath before they enter the heated die or by injecting resin through ports in the heated die; and (iii) the fibres are well distributed in all cross-sections. It is not usual to have fibre weight fractions (fwf) greater than 60%, although uncomplicated sections, such as small-diameter rods with unidirectional aligned fibres, have been manufactured with 70% fwf. If the pultruded composite is to be used for rehabilitating structures, a roughened surface for increased bonding capability is required on the side of the pultruded unit to be bonded to the soffit or sides of the structural member. This roughened surface can be achieved by using a peel-ply on that side; however, if this system is used, both sides of the pultruded composite plate would generally have a peel-ply fabricated onto them to maintain symmetry within the plate. A peel-ply is a layer of nylon or polyester fabric incorporated onto the surface of the composite during manufacture. The peel-ply is stripped from the pultruded surface immediately prior to bonding to the adhered thus providing a clean, textured surface to the composite unit. Furthermore, to provide resistance to hostile environments, a resin-rich exterior surface to the pultruded section can be fabricated using a surface veil, which is also incorporated into the structural component at the time of manufacture. The limiting factor on the size of the unit and the complexity of the cross-section is the pull force required to draw the pultruded section through the die; the more complex the section the greater will be the force due to friction of the fibre–matrix material within the die. Hydraulic pulling force systems up to 50 t on sections of 2.5 m by 275 mm thick are currently in operation in the USA. Carbon, aramid and glass fibres and epoxy, vinylester and polyester materials have all been used for the production of pultruded units. The epoxy polymers are probably the most difficult to pultrude but do have low shrinkage during polymerisation (3–4%). The vinylester polymers have a shrinkage value of (6–10%) and the polyester polymers have a large shrinkage during polymerisation of (12–19%). Further information may be obtained from Starr (2000) and Hollaway and Head (2001). **Figure 2** illustrates the pultrusion technique.

Companies that provide details of geometries and mechanical properties of pultruded profiles are Fibreforce (UK), Fiberline (Europe), Creative Putrusions, Strongwell and Bedford (USA).

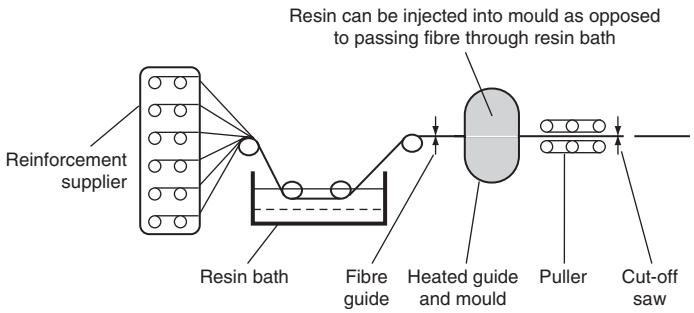


Figure 2 Diagrammatic representation of pultrusion technique

The pultrusion technique is utilised in civil engineering to manufacture and construct 'all composite' structural members and bridge decks, to rehabilitate and to retrofit structural beams. It is also used as the reinforcing rods in near-surface mounted constructions, as internal reinforcement to concrete construction and in bridge enclosures and fairings.

The filament winding technique

This technique is used to manufacture pressure pipes and to undertake wrapping of columns (Chapter 58). There are two different winding methods:

1 the wet winding;
2 the pre-impregnated winding.

The wet winding method consists of continuous strands or roving of dry fibres from a series of creels which are passed through a bath of 'cold cure' resin, cold-curing agent, pigments and UV retardants on to a 'pay-out eye' which is mounted on a moving carriage along the length of a constant-speed rotating mandrel. The roving fibre delivery system reciprocates along the length of the mandrel and is controlled relative to the rotation of the mandrel to give the required fibre orientation. The speed of reciprocation and rotation are synchronised to hold a preset winding angle typically between 7° and 90°. The machine has the ability to lay fibres in any direction and to employ as many permutations of movements as is required by the structural design. After winding, the filament-wound mandrel is subjected to curing and post-curing operations during which the mandrel is continuously rotated to maintain uniformity of resin content around the circumference. After curing, the product is removed from the mandrel either by hydraulic or mechanical extraction. **Figure 3** shows a schematic representation of the filament winding technique.

The pre-impregnated winding consists of passing pre-impregnated fibres over a hot roller until its surface is tacky to the touch; they are then wound on to the rotating mandrel.

The wet winding is the method usually used and has several advantages over the pre-impregnated winding. These are low material cost, short winding time, and the resin formulation can be readily varied to meet specific requirements.

The Xxsys Technologies' method is a site filament winding technique. It was first developed in Japan in the early 1990s and then in the USA by Xxsys Technologies, Inc. San Diego, California for seismic retrofitting and strength restoration of concrete columns using continuous carbon fibre. It involves continuous winding of wet fibres under a slight angle around columns using a robot; this results in good quality control and rapid installation. The Xxsys carbon fibre composite jackets are installed with a fully automated machine called Robo-Wrapper™ and portable oven for curing. The technology associated with the technique is based upon the filament winding of prepreg carbon fibre tows around the structural unit thus forming a carbon fibre jacket; currently, the structural unit to be upgraded would be a column. The polymer is then cured by a controlled elevated temperature oven and can, if desired, be coated with a resin to match the existing colour of the structure. An advantage of this automated process is that the carbon fibre prepreg is impregnated with the polymer under factory-controlled conditions, providing good quality control and as a consequence a high strength-to-weight ratio. The equipment is erected on site with minimum disturbance to traffic and the whole operation is undertaken in minimum time; the latter will, however, depend upon the size of the job. The carbon fibre jacket which is eventually formed around the column will increase the shear capacity of the column and will confine the concrete and greatly enhance its ductility in the flexural plastic hinge region. Furthermore, it will provide lap splice clamping and will prevent local buckling of the vertical reinforcement. For corrosion-damaged columns, the jacket restores shear capacity and will prevent spalling of the cover concrete.

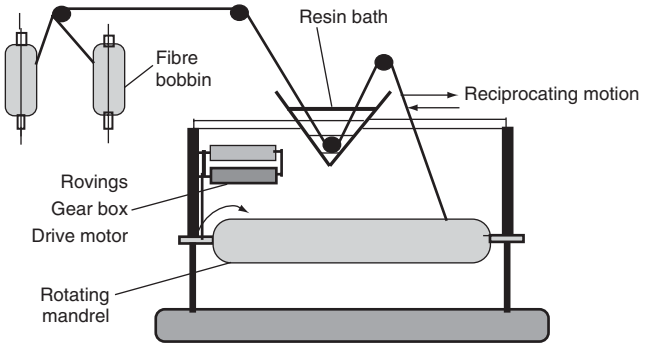


Figure 3 Diagrammatic representation of the filament winding technique

Resin transfer moulding process

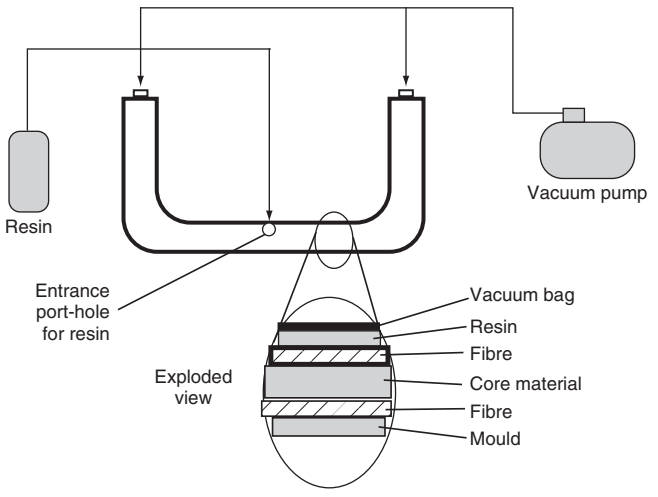
This is a low-pressure, closed mould, semi-mechanised process. The method enables fabrication of simple low-to-high performance articles in varied sizes and profiles. The resin transfer moulding (RTM) process has been successfully used to mould complex 3D shapes. In RTM, several layers of dry continuous strand mat, woven roving or cloth are placed in the bottom half of a two-part closed mould and a low-viscosity catalysed liquid resin is injected under pressure into the mould cavity, which is subsequently cured. Instead of using flat reinforcing layers such as a continuous strand mat, the starting material in RTM process can be a 'preform' that already has the shape of

the desired product. The potential advantages of RTM can be summarised as rapid manufacture of complex, high-performance structures with good surface finish on both sides, design flexibility and capability of integrating large numbers of components into one part. The technique is sometimes used to manufacture highway street furniture.

There are three main RTM moulding methods for the production of FRP industrial units, all of which are a variation of the general RTM method; these are the resin infusion under flexible tooling (RIFT), the Seemann composites resin infusion moulding process (SCRIMP), the vacuum assisted resin transfer moulding (VARTM) and the (TERTM)^{TM} methods. Of these, the RIFT method is the one that has been used in construction.

The resin infusion under flexible tooling (RIFT) process

The RIFT process is one in which one of the solid tool faces of the RTM method is replaced by a flexible polymeric film. Potentially a very clean and an economic composites manufacturing method, the process draws resin into a dry reinforcement on an evacuated vacuum bagged tool using only the partial vacuum to drive the resin. It reduces the worker contact with the liquid resin while increasing the component mechanical properties and the fibre content by reducing voids in the composite compared to the wet lay-up process. For higher-performance composites, RIFT offers the potential for reduced tooling costs where matched tooling (RTM or compression moulding) is currently used. In the RTM technique the dimensions of the component are defined by the separation of the mould faces, whilst in the RIFT process the thickness of the part is a function of the pressure history during the process. This technique has been used to upgrade cast iron girders. **Figure 4** illustrates the RIFT technique.



Manufacture of a sandwich construction by the RIFT technique

Figure 4 Schematic representation of the RIFT technique

Mechanical properties of FRP composites

Tensile properties

As the reinforcing fibres are generally designed to be loaded along their length, and not across their width, the orientation of the fibres creates highly 'direction-specific' properties in the composite. This 'anisotropic' feature of composites can be used to good advantage in designs, with the majority of fibres being placed along the orientation of the main load paths. This minimises the amount of parasitic material that is placed in orientations where there is little or zero load. Furthermore, the properties of the composite are highly dependent upon the type of fibre used. As has been discussed in Chapter 51, the three fibres used in civil engineering are the carbon, the aramid and the glass fibre and their stiffness and strength in tension have been given in Chapter 51, Table 3. Clearly the greater the stiffness of the fibre, the greater will be the stiffness of the composite. The value of the compressive modulus of elasticity of FRP materials is generally lower than their tensile modulus value. Test samples of unidirectional fibres (UD) in a vinylester polymer containing 55%–60% weight fraction of continuous fibres in a vinyl-ester resin have compressive modulus of elasticity values of approximately 80%, 100% and 85% of the tensile value for glass-fibre-reinforced polymers (GFRP), aramid-fibre-reinforced polymers (AFRP) and carbon-fibre-reinforced polymers (CFRP) respectively. Subramaniyan *et al.* (2003) have shown that by adding nanoclays to the polymer the compressive strengths of GFRP composites do increase. **Figure 5(a)** and **(b)** show typical tensile and compressive stress–strain characteristics, respectively, of a UD prepreg laminate.

Compressive properties

The integrity of both component parts of the composite is far more important under a compressive load than under a tensile one. Furthermore, local resin and interface damage caused by compressive loading leads to fibre instability which is more severe than the fibre isolation mode which occurs in tensile loading. The mode of failure for FRP composites subjected to longitudinal compression will depend upon the type of fibre, the fibre volume fraction, the type of resin and may include fibre micro-buckling, transverse tensile failure or shear failure. The compressive strengths of CFRP and GFRP composite materials increase as the tensile strengths increase, but the aramid fibres in AFRP composites exhibit non-linear behaviour in compression at a relatively low level of stress (**Figure 5b**). Many factors influence the compressive response of composite materials. On a micro-structural level, the presence of local inhomogeneities and defects, which are difficult to characterise, influence the failure mechanisms that will dominate the response of a composite in compression

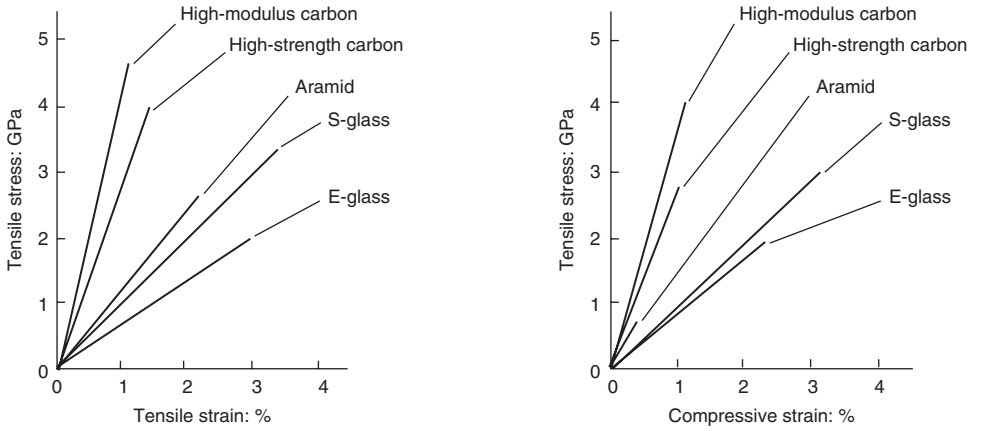


Figure 5 The tensile and compressive stress–strain characteristics of a U/D prepreg laminate

more than any other state of stress. Consequently, manufacturing plays a significant role in determining the compressive response of composite materials, and will play an even greater role as the section thickness of the composite increases. Constituent properties, laminate orientation, specimen geometry, method of loading, fibre waviness, voids and stress concentrations all have been shown to play a role in determining the predominant failure mode governing compression failure. These failure modes include global Euler buckling, micro-buckling, transverse tension, fibre kinking, fibre compression failures, matrix compression failures or delamination. Observations signify that there is no single definition of compression failure strength. The two extremes are global elastic Euler buckling and material compression failure. Between these two extremes lies a transition region in which the state of stress that exists is a combination of bending-induced stresses and compressive stresses.

Impact resistance

FRP composite materials under impact force is a complex subject and as such is a major concern in the design of FRP composite structural members; it is well known that laminated composites perform very poorly when subjected to transverse impact loads. Furthermore, uncertainties about the dominant mechanism of damage make it difficult to ascertain the effect of an impact event on the performance of an FRP structural member. Compounding these considerations is the fact that, in some cases, the damage will not be visible, which could lead to components with considerable damage being allowed to continue in service. These damage modes are affected by the properties of both the impactor and the composite. A single event can produce several different damage modes simultaneously. Due to the variability of FRP laminate properties, which is primarily due to inconsistencies in their fabrication processes, it is not possible to compare the results from different studies even when the specimens are theoretically identical. The primary focus of research into this topic has been aimed at exploring the impact behaviour of unstressed plates, the basic mechanics governing this behaviour has been given in several publications, Hyung Yun Choi (1992), Soutis and Curtis (1996), Tien-Wei Shyr and Yu-Hao Pan (2003), Chang *et al.* (2004), Wang *et al.* (2005).

Any initial damage caused to an FRP composite under an impact force consists of matrix cracking and fibre buckling; this is largely a linear-elastic relationship. No significant reduction in residual strength is caused by these two damage types. During a more severe impact force the damage might involve the delamination or debonding of individual laminae of the composite. It has been shown that subsequent to the initial 'force threshold' known as the 'delamination threshold load', the delamination area, indicated by an ultrasonic C-scan, increases linearly with the degree of impact energy. When the upper energy boundary is reached, the delamination which occurs at the interface between plies of different orientation does not increase further but the fibre damage accumulates (Abrate, 1998). There are three components that affect the size of a delamination: the difference between the elastic constants (E_{11} and E_{22}) of a composite, the thickness of the composite and its deflection. Depending upon the type of loading (tension, compression or bending), the damage caused to the various laminae layers of the composite during an impact event may be one or more of three types, i.e. matrix cracking, delamination or fibre fracture. The least important of these fractures is caused by matrix cracking but this can initiate delamination at the ply interface which does degrade the mechanical and physical properties of the composite.

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There are various degrees of failure delamination of the impacted laminae of the FRP composite. In symmetrical laminates subjected to:

- Uni-axial or bending loads – the separation of the laminates can result in the individual laminates becoming unsymmetric and thus causing shear and torsion stresses to be set up in the composite.
- Pure tension loads – this situation is not as critical as when the composite is subjected to compression or bending but will reduce the ability of it to transfer load from damaged layers to undamaged layers through interlaminar shear stresses.
- Bending loads – this situation is more pronounced than when tensile loads are applied as there is no composite action between the individual laminae, there is no strain compatibility and the moment of inertia of the composite is reduced.
- Compression loading – this situation is one of stability and depends upon the size and location of the delamination; if it is severe and close to the centre of the composite it will behave similarly to two separately loaded members. The effect will be negligible if the delamination is small.

Site composites in service are generally under external loads at the time the impact load is applied. Sun and Chen (1985), Kelkar *et al.* (1997), Chiu *et al.* (1997) and Nettles (1998) undertook some early work exploring the response of pre-tensioned composites subjected to impact. The conclusions of the last two researchers was that unidirectional or bidirectional composites under a tensile or a compressive load, were stiffer than the unstressed ones with the result that the peak loads were increased, contact time was decreased and the deflection was also decreased. The pre-tensioned laminates had larger delamination areas and more (back) face damage than the unstressed ones. The latter damage is likely to be due to the combination of bending from impact, stress from the pre-loading and membrane stresses. Martin and Hansen (1996) and Chiu *et al.* (1997), also undertook tests on pre-loaded compressive composites and observed that the effective stiffness of the composite reduced the peak load but increased the contact time, the deflection and the delamination area.

Delaminations are thought to be dependent upon fracture toughness of the polymer material and attempts have been made to improve delamination resistance by the use of toughened polymers and interleaves. Ozdil and Carlsson (1992) and Aksoy and Carlson (1992) showed that toughening interleaves can considerably improve interfacial fracture toughness in composites. Furthermore, Masters (1989), Rechak and Sun (1990) and Lee and Chang (1993) showed that interface toughening can improve significantly the impact resistance of composites to low-velocity impact. Impact damage can pose particular problems when using high-stiffness fibres in very thin laminates. In some structural members, where cores are used (e.g. sandwich members), the face composites can be less than 0.3 mm thick, although other factors such as weave style and fibre orientation can significantly affect impact resistance. In impact-critical applications carbon fibres are often found in combination with other fibres; this can be in the form of a hybrid fabric where more than one fibre type is used in the fabric construction.

Blast strengthening

Recent world events such as the former bombings in Northern Ireland and the bombings in London (2005), Madrid (2004), Istanbul (2003), Bali (2002) and New York (2001) have highlighted the susceptibility of many civilian structures to terrorist attacks. Explosives directed towards vulnerable structures can cause considerable damage and loss of life. Consequently there is now a desire to increase the blast resistance of many types of existing and new structures.

The response of a structure to blast loading is complicated and depends on the mass, geometry and material properties of the structure as well as the pressure magnitude of the impact loading. The materials of concrete, steel and FRP have different properties in strength and ductility when the loading rate changes. It is commonly accepted that, in terms of failure mechanism, reinforced concrete columns are among the most vulnerable elements during abnormal loads, such as blast loads and seismic excitation. Reinforced concrete columns and walls are economically retrofitted using FRP composite wraps to resist blast effects. As FRP composites exhibit linear behaviour up to failure, this property will dissipate energy through strain and will allow relatively large deformations without failure of the existing structure. Muszynski (1998) investigated CFRP composite strengthened walls which were subjected to blast loading and were damaged by the debonding of the FRP composite or by its tensile failure at the mid-height of the wall; after the loads were released there was less residual displacement compared with that of the unstrengthened control walls. Crawford *et al.* (2001) suggested that CFRP sheets were appropriate for some blast strengthening situations. Carney and Myers (2003) studied the connection between FRP composite strengthened infill walls and the concrete boundary; they examined two connection details. Increases in strength and strain energy of the order of two to three times were observed for the walls with the boundary connection compared to the walls with no connection. These studies verified that the characteristic of FRP retrofits prevents the scatter of loose fragments and debris; this is an important safety factor for occupants of building. Abou Zeid and El-Hacha (2007) evaluated the effectiveness of rehabilitating reinforced concrete columns using an externally bonded FRP jacketed system under the effect of a blast loading. They concluded that the rehabilitation system resulted in a significant improvement in the blast performance by comparison with the response of the existing non-ductile building.

The in-service properties of polymer composites

The long-term durability of FRP composites under physical and mechanical loading

Although the short-term benefits of the use of FRP composites are well documented and proven, the long-term durability of the material is less well understood, and it is a major obstacle to the validation of FRP composite technology for the strengthening of concrete structures. Karbhari *et al*. (2000) have stated that often durability is erroneously related only to environmental-based degradation, whereas they have suggested that the term should better be defined as 'the ability of a material or structure to resist cracking, oxidation, chemical degradation, delamination, wear, and/or the effects of foreign object damage for a specific period of time under the appropriate load conditions, and some specified environmental conditions'.

Degradation of FRP materials is not unique; indeed all engineering materials undergo mechanical and in-service changes in different ways due to environmental effects. Composite materials do offer some significant durability advantages over the more conventional construction materials but the environments which do cause the major durability concerns for FRP composites in civil engineering practice are:

- moisture, aqueous and chemical solutions (alkaline environments);
- corrosion resistance;
- fire behaviour;
- thermal effects;
- ultraviolet radiation.

In addition, the FRP material may have to function under long-term loading conditions to resist:

- fatigue; and
- creep.

Moisture, aqueous and chemical solutions

The durability of FRP composites does require further research to be undertaken to enable a greater understanding of the characteristics of the FRP material when loaded over long periods of time. The main problem in undertaking detailed analysis of the durability properties is the length of time involved in gathering the relevant information. In construction there are many different polymers on the market and some of these have been modified by chemists over the years to improve their physical and mechanical performance. In addition, additives are on occasions incorporated into cold-setting polymers to enhance curing. Each time these polymers are changed/modified the durability will be affected.

Corrosion resistance

Unlike metals, composites do not 'rust'; this makes them attractive in applications where corrosion is a concern. This has led to an intense interest in composite reinforcing (rebar) bars and grids, as well as cables for pre- and post-tensioning, and for cable stays. Composite material components are used in applications where standard metallic components incur high maintenance costs due to corrosion and its effects. Examples where composites are used for their corrosion resistance is in the upgrading of reinforced concrete (RC) and steel structures where the material is exposed to salt solutions during winter snowstorms, marine waterfront structures, cladding panels, pipelines and walkways in harsh environments.

Fire behaviour

The polymer component of the composite used in the construction industry is an organic material and is composed of carbon, hydrogen and nitrogen atoms; these materials are flammable to varying degrees. Consequently, a major concern for the construction engineer using polymers is the problem associated with fire. Most building structures must satisfy the requirements of building codes which relate to the behaviour of structures in a fire. A measure of fire ratings for buildings refers to the time available in a fire before the structure collapses; a series of fire codes have been given in the appendix to this chapter. However, the major health hazard derived from polymer and composites in a fire accident is generated from the toxic combustion products produced during burning of materials. The degree of toxicity generated depends on the phase of burning of the fire including: oxidative pre-ignition, flaming combustion or fully developed combustion and ventilation controlled fires. With the application of a wide range of polymer composites in buildings, health hazards from exposure to airborne thermal decomposition products is of concern. This smoke stream can contain a mixture of evolved gases, vapours, free radicals, and particulates which may present a toxic and hazardous situation (Gandhi and Lyon, 1998; Purser, 2002). Smoke toxicity plays an important role during fire accidents in buildings, where the majority of people die from smoke inhalation. Many of the building applications have either prescribed or performance requirements for fire but few actually require control of the toxicity. This has arisen because of the complexity in identifying individual chemicals and their associated toxicity. Hence, it relies on controlling the fire growth and spread in order to limit the generation of toxic products. Improved methods of assessment need to be developed if toxicity is to be included as part of the fire hazard risk identification. Current ASTM

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standards which are concerned with the measurement of gasses present or generated during fires and their smoke density are ASTM E 800-01, ASTM E1678-02 and ASTM E176-04. The British Standard Codes which are concerned with smoke toxicity in fire hazards and risk assessment are BSS 7239-88, *the Boeing Toxicity Test* and BSS 179-03, *The Use of Bench-scale Toxicity Data in Fire Hazards and Risk Assessment*.

Sufficient research and implementation has been conducted for the development of various design codes and guidelines for externally bonded FRP composites used in the rehabilitation of structural beams and columns associated with bridges (ACI. 440.2R-02, 2002; CSA. S806-02, 2002; *ISIS Design Manual*, No. 4, 2001; The Concrete Society Technical Report 55, 2000; The Concrete Society Technical Report 57, 2003). Generally, the design for the rehabilitation of bridges is such that if the upgrading composite plate is destroyed the bridge will not collapse; in these structures, therefore, fire is not a primary concern. However, for externally bonded FRP systems to access the full range of potential applications, including strengthening and repair of *interior* building components, the issue of the fire resistance of FRP materials and externally bonded systems must be addressed. FRP composites used in construction are susceptible to combustion with increased flame spread and toxic smoke evolution. Furthermore, as the temperature rises above the glass transition temperature (T_g) of the polymer and the adhesive, which varies from 65°C to 82°C for externally bonded systems (ACI. 440.2R-02, 2002), the composite loses strength and stiffness. Consequently, it might be difficult to maintain the effectiveness of externally bonded FRP strengthening systems during a fire if left unprotected. It is possible to achieve a satisfactory fire performance provided they are appropriately designed with adequate insulation.

Ways of improving the fire resistance of polymer composites is to incorporate additives into the resin formulations but in so doing an impurity is added to the polymer and some mechanical and/or in-service property of the polymer may be compromised. Nano-clay particles will give some protection against fire and may be added to the pristine polymer, but the process is complicated and at present is expensive for the civil engineering industry (Hackman and Hollaway, 2006). The chemical structure of the polymer could be altered, thereby modifying the burning behaviour and producing a composite with an enhanced fire property. Nevertheless, the modification of the polymer can only aid the fire resistance of the composite to a certain degree; eventually fire will damage composites and indeed all civil engineering materials. Virtually all composites used in structural engineering will have high fibre volume fractions and thus the rate of progress of the fire through the composite is slow; the carbon, glass or aramid fibres do not burn.

Deuring (1994) and Blontrock *et al*. (2001) have investigated the fire resistance of FRP composites bonded to RC beams and Saafi and Romine (2002) have undertaken limited laboratory experimental work on wrapped cylindrical columns after exposure to elevated temperatures. Bisby *et al*. (2004) undertook a numerical and experimental programme to investigate the fire performance of FRP-wrapped full-scale reinforced concrete columns in order to provide fire-design guidance. The numerical modelling undertaken in this study is capable of predicting the thermal and structural response of an FRP-wrapped concrete column under exposure to a standard fire.

Thermal effects

Coefficient of thermal expansion

The coefficient of thermal expansion of polymers is much higher than the fibre component of the fibre–matrix composites: the former material's value is of the order of $100 \times 10^{-6}/°C$. The equivalent value for glass, aramid and carbon fibre components are in the region of $10 \times 10^{-6}/°C$, $-2 \times 10^{-6}/°C$, and $-0.9 \times 10^{-6}/°C$ respectively, depending upon the type of specific fibre. The thermal expansion of an FRP composite system is reduced from the high value of the polymer to a value near to that of conventional materials; this reduction is due to the stabilising effect that the fibres have on the polymer. The actual value will depend upon the type of fibre, the fibre array and the fibre volume fraction of the composite. In addition, the value of the coefficient of thermal expansion will vary with the temperature and the temperature range into which the composite is placed. Furthermore, the degree of cross-linking of the polymer will also influence the rate of thermal expansion.

Thermal conductivity

This property is particularly important when FRP composites are exposed directly to the sun's rays; an example where this is particularly relevant is in FRP bridge decks (Chapter 58). As the thermal conductivity of polymers is low, they are good heat insulators (Chapter 51). To enable a prediction of the failure mechanisms and the 'fire resistance limit' of a superstructure of a bridge under extreme thermal loading conditions, Alnahhal *et al*. (2006) undertook a finite-element thermal-stress analyses, and the results were verified by field results provided by the New York State Department of Transportation.

Ultraviolet radiation

FRP composite structures designed for weather exposure are normally fabricated with a surface layer containing a pigmented gel coat or have an ultraviolet (UV) inhibitor included as an additive to the composite matrix. Both methods provide protection to the underlying material by

screening out UV rays and minimising water absorption along the fibre–resin interface.

The long-term durability of FRP composites under fatigue and static loads

Fatigue characteristics of advanced composites

With the introduction of lightweight materials into the construction industry, such as FRP composites with high strength and stiffness, problems relating to corrosion and fatigue have become important, particularly when used to rehabilitate and extend the service life of existing structures. Corrosion is a major deterioration problem in marine and bridge structures which is exacerbated when these structures are subjected to oscillating loads which cause fatigue; this loading significantly reduces the expected life of the structure. Even in less corrosive marine environments fatigue will eventually cause the degrading or failure of the mechanical properties of a fibre–matrix composite material after it is exposed to the repeated application of stress. Furthermore, a complex failure mechanism and extensive damage can be caused to this anisotropic material. Unidirectional continuous fibre composites loaded in tension parallel to the longitudinal fibre have good fatigue properties, which are essentially linear to failure. However, if the composite contains off-axes plies, various damage mechanisms can occur under loading and these will redistribute the internal load and cause the stress–strain response to become non-linear. Polymeric composite materials experience progressive fatigue degradation due to failure of the fibres, fibre stacking sequence and type of fatigue loading. Under this type of loading the damage development is similar to that under a static loading with the exception that the fatigue loading at a given stress level will cause additional damage to the composite; this damage will be dependent upon the cyclic frequency. Kim (1987), Beaumont *et al.* (1991), Echtermeyer (1991) and Plumtree and Shen (1991), have discussed the failure mechanisms in greater detail and the fatigue failure characterisations have been discussed in Kim (1987). There is much less information available on the compressive fatigue of composites. This is mainly due to the complexity of the test procedure including the need to support the specimens undergoing global microbuckling combined with limitations imposed on specimen geometry by the anisotropic nature of the materials.

There are four basic failure mechanisms of polymeric composites under fatigue loading. These are:

- fibre breakage interface debonding;
- matrix cracking;
- interface shear failure with fibre pull-out;
- brittle failure.

Harries and Minnaugh (2006) have investigated the fatigue behaviour of externally bonded steel-fibre-reinforced polymer (SFRP) for rehabilitation of RC beams and have compared these results with those of a similar strengthening system using CFRP composites; the steel wire in the SFRP material has been described in Hardwire (2002). The results have indicated that SFRP is more flexible than CFRP, resulting in a greater application width required and effecting a comparable strengthening. This resulted in decreased interfacial stresses and improved fatigue behaviour with no evidence of debonding in the relatively high stress ranges tested in the programme of work. The previously observed deleterious effects of fatigue loading on bonded CFRP systems appeared less significant in bonded SFRP systems.

Further examples of fatigue loading on the rehabilitation of structures are given in Chapter 58.

The creep of fibre–polymer composites under an external load

The polymer component of an advanced composite is a visco-elastic material and therefore the composite material will creep. The creep characteristics are highly dependent on the resin properties, the fibre types and the environmental conditions such as temperature and humidity; carbon, glass or aramid fibres have very low creep values. Consequently, the fibres have a stabilising influence upon the creep characteristics of the advanced composite material and its actual creep value will be dependent upon the fibre volume fraction and the orientation of the fibres.

The deformational behaviour of an advanced polymer composite is dependent upon the way in which the composite is subjected to stress and strain and these latter values will differ under different short-term conditions of testing. It is, therefore, imperative that a standard test procedure to obtain mechanical behaviour of composite materials is adhered to. The mechanical behaviour of composite materials is more realistically established by applying constant loads over longer periods of time; these investigations are defined as creep tests. The tests produce curves of elongation against time at different stress levels and although they are not able to produce data that may be converted directly into stress-strain curves, constant time sections through families of such creep curves have been used to produce isochronous stress–strain curves.

McMurray and Amagi (1999) describe the time and temperature behaviour of fibre-reinforced epoxy polymer composites when used for long periods and under high temperatures. The time and temperature dependence of flexural strength was examined by creep and fatigue testing. The former tests were carried out at various temperatures

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below the glass transition temperature (T_g) and the latter tests were carried out at various stress ratios, temperatures below the T_g and two frequencies. The time–temperature superposition principle held for the flexural creep strength of this material. The researchers proposed a method to predict flexural creep strength based on the static strength master curve and the cumulative damage law is proposed.

Monitoring durability of polymer composites

There are two possibilities for obtaining information on the durability of polymers, these are as follows.

Monitoring FRP composite structural components

With the increase in the number of FRP composite structural members being constructed currently, and a sparse knowledge of the long-term property characteristics of composite materials, it is advantageous to monitor the performance of such structures with time. This is the most relevant durability test to establish the long-term characteristics of polymer composites used in civil engineering construction. This procedure might involve: (i) test loading the structural system at certain periods of time after construction to obtain stress and deflection values against time under specified external loading; such tests would characterise the overall behaviour of the structural system; (ii) visual inspections of the structure, at specified times, for any signs of distress or deterioration of the material from the point of view of environmental degradation or from exposure to long-term application of loads which might influence the creep rate of the polymer component of the composite; (iii) mechanical and physical/chemical material tests on specific areas of the exposed FRP structure to investigate any possible degradation effects between the component parts of the FRP material. The disadvantage of this method in obtaining information is the length of time involved.

Undertaking accelerated testing in the laboratory

To obtain the durability characteristics of a composite as quickly as possible, sometimes test specimens are exposed to an accelerated test regime which generally involves the specimens being subjected to an environment many times more severe than that which would be experienced in practice. In addition, these test samples are sometimes exposed to elevated temperatures to further increase degradation rapidly. High temperatures are not relevant for most civil engineering polymer composites that operate at normal environmental conditions. Furthermore, the degradation mechanism in FRP materials at high temperatures is different to those under the lower practical temperatures. It will also be realised that as the environmental temperature rises towards the glass transition temperature of the polymer, the latter will lose some of its stiffness and strength with the result that the investigation will not be analysing the original material. The accelerated test procedures for obtaining durability data under one environmental situation, will generally not be equivalent or relevant to the more gradual degradation effect had the environment been applied in a less rigorous manner. Moreover, materials used in construction would normally be exposed to many different environments acting simultaneously but in a less harsh way, and each environment possibly having an effect upon the other. It can be seen that the method of accelerating the degradation of polymer composites should be used with caution. The accelerated test method can be used satisfactorily if the test objective is to compare the behaviour of one material with another under identical environments. Examples where completely inappropriate accelerated tests were undertaken and the wrong conclusion drawn have been given in Hollaway (2007).

Many field surveys have been discussed by Hollaway (2007), which are currently being or have been undertaken throughout the world. The results from these investigations are revealing and have provided interesting discoveries regarding the long-term resistance of FRP composites to the natural environments to which civil engineering composites are exposed. For instance, at the end of the last decade a number of laboratory accelerated tests indicated that the sulfate in the cement used in concrete was particularly harmful to FRP rebars. Hollaway (2007) has discussed these accelerated test results and due to the importance that the author attaches to the understanding and interpretation of accelerated test results, the test descriptions given in that chapter and in Hollaway (2008a,b) have been reproduced here. The work reported in Bank and Gentry (1995), Sen *et al.* (2002) and Bank *et al.* (1998) exposed GFRP to a simulated concrete pore water solution of high pH values and elevated temperatures up to 80°C, and have indicated that there is a decrease in the tensile, shear and bond strengths; these results would suggest that there is a case for not using GFRP rebars in concrete (Uomoto, 2000). However, Tomosawa and Nakatsuji (1997) have shown that after 12 months, exposure to alkaline solutions at temperature between 20° and 30°C, and Clarke and Sheard (1998), likewise, after two years' exposure to a tropical climate on a test platform off the Japanese coast, have reported that there had been no material or physical deterioration to the GFRP composite. Furthermore, Sheard *et al.* (1997) reported that the overall conclusions of the work of the EUROCRETE project were that GFRP is suitable in a concrete environment. It is possible to use an extrapolated process on data obtained from short-term field tests. Greenwood (2001) compared extrapolated data from a three-month period with field data obtained after three years and concluded that realistic estimations can be obtained at least

within this period. In the USA, the Federal Highway Administration (FHWA, 2006) suggests an environmental reduction factor of 0.65 to be applied to the mechanical properties to account for the degradation over time, for a service life of 75 years. Currently where possible the in-service conditions should be monitored by testing with fibre-optic sensors, so that the long-term performance is ascertained. Alampalli (2005) reported on the performance of an FRP superstructure (8 m long and 10 m wide deck) built in 1998. Beyond the visually satisfactory state, he reported that the observed deflections were 2.5 times lower than those predicted at the design stage, thus questioning the degradation factors currently being used. In addition he reported that the adhesive joints had performed as intended.

Future work on FRP materials

There are still problems which confront the designer of FRP composite materials in the civil infrastructure projects; these are:

1. The repeatability of properties of mass-produced composite products and the compliance with construction tolerances. Coordinated efforts are required to achieve significant advancements in the development of composite material specification and quality-control guidelines. The establishment of a programme to qualify specific construction and repair composite materials is essential to expedite the use of composites in the infrastructure. The repair of infrastructure construction utilising composites and hybrids may be a cost-effective alternative to conventional technologies but it is necessary to develop techniques to evaluate their in-service performance to ensure continuity integrity. Ultrasonics, acoustic emission and infrared thermography test equipment are now used in areas of defect detection in structural composites. To address these issues there is a need for coordinating research activities at an international level to avoid duplication and to build on each other's progress.
2. The long-term durability of composites. There is a strong need for continuous monitoring of durability data and, as a result of this, a gradual build-up of a data bank of durability information, codes and databases.
3. The evaluation of composites in the infrastructure may be considered as a three-fold task that encompasses the elaboration of design guidelines and specifications, the understanding of the long-term performance and the development of construction and repair technologies. These tasks are progressing well, as is evident in the applications of composites in (i) bridge decks, (ii) rehabilitation of beams and retrofitting of concrete columns, (iii) the use of FRP composite dowel bars between sections of concrete road slabs; this environment is highly corrosive to steel dowel bars, (iv) the gradual construction of gantries and pedestrian foot bridges over road and railways and (v) the engineered wood products. These examples must and do contain a cost competitiveness with the more conventional civil engineering materials.

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Chapter 53

Testing and evaluation of polymer composites

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A comprehensive hierarchy of mechanical test methods has been developed for polymer composites to accommodate the diverse range of fibre formats encountered. This section provides an overview of the current status of test methods for measuring laminate and constituent (fibre and matrix) mechanical properties.

Introduction

Engineering structures are often unavoidably complex, consisting of a number of geometric features, such as free and curved edges, ply termination, thickness changes, and bonded or bolted joints. Until recently, the general tendency has been to use two-dimensional (2D) stress analysis to evaluate structural response to three-dimensional (3D) loading configurations. Three-dimensional finite-element analysis, now in common use, frequently requires a full complement of in-plane and through-thickness (T-T) properties, especially for those 'difficult but real' aspects of design where the composite, to perform its function, has to be shaped or connected to other components. Designers/engineers require both short-term and long-term data on both coupon and structural components. In response to these demands, a comprehensive hierarchy of mechanical tests methods has been developed for polymer composites to accommodate the diverse range of fibre formats encountered. This chapter provides an overview of the current status of test methods for measuring laminate and constituent mechanical properties (see **Table 1**).

The chapter concentrates on mechanical property test methods supporting design of composite components. A good summary of other applicable test methods is given in the annexes of EN 13706.2: *Pultruded Profiles – General Aspects and Test Methods*. A full listing of composite material standards is given in Chapter 7.

Tension

Fibre bundles

Several test methods exist for measuring the tensile strength and longitudinal modulus of single fibre filaments and fibre tows or rovings (i.e. untwisted bundle of continuous parallel filaments). These methods are usually intended for yarns having a diameter less than 2 mm (typically 0.5 to 0.8 mm), or a linear density lower than 2000 tex (g/km). Fibre tow methods include testing of both unimpregnated (i.e. loose or dry) and impregnated rod rovings (**Figure 1**). ISO 3341 specifies a method for the determination of the tensile breaking force and failure strain of unimpregnated glass fibre rovings, whereas ISO 9163 allows for both unimpregnated and impregnated fibre tows. ISO 10618, ISO 11566 and ASTM D 4018 specify tensile test methods for carbon fibre tows.

Static fatigue testing of resin-impregnated strands is a convenient method of obtaining basic strength–time data in a relatively short period of time (accelerated ageing) – see Cowking *et al.*, 1981; Bader *et al.*, 1993; and Broughton and Lodeiro, 2000. The test can be used to evaluate quantitatively the combined effects of applied stress and aggressive environments (**Figure 2**). The small diameter (0.5–1.0 mm) of the specimen enables rapid equilibrium with the surrounding environment.

Polymers

Tensile specimens typically consist of a waisted section with parallel sides (dumbbell) to facilitate strain measurements and to ensure failure occurs within the gauge-section away from the gripped ends. ISO 527-2, which specifies test methods for determining the tensile properties of plastics and PMCs, includes several dumbbell specimen configurations. No end tabs are required. Testing and data reduction are relatively straightforward. The tensile stress is simply the applied load divided by the cross-sectional area of the specimen gauge-section (i.e. $\sigma = P/A$). Contact (e.g. strain gauges, extensometers) and non-contact (e.g. video and laser scanning extensometry) techniques can be used to measure strain.

Composites

ISO 527-4 and 5 provide tensile testing specifications for determining the ultimate tensile properties of multidirectional and continuous aligned (longitudinal (0°) and transverse (90°)) laminates, respectively (see **Figure 3**). Specimens are typically 250 mm in length with a 150 mm gauge length. The width of longitudinal and transverse unidirectional laminates is 15 mm and 25 mm, and thickness 1 mm and 2 mm, respectively. The width of multidirectional laminates is 25 mm and thickness 2–10 mm. The ends of the

Load type	Elastic moduli	Poisson's ratio	Strength	Failure strains
Tension	E_{11}, E_{22} [E_{33}]	$\nu 12$, $\nu 21$ [ν_{13}, ν_{31}, ν_{23}, ν_{32}]	S11T, $S22_T$ [$S33_T$]	$\varepsilon 11$, ε_{22} [ε_{33}]
Compression	E_{11}, E_{22} [E_{33}]	$\nu 12$, $\nu 21$ [ν_{13}, ν_{31}, ν_{23}, ν_{32}]	S11C, S22C [S33C]	$\varepsilon 11$, ε_{22} [ε_{33}]
Shear	G12 [G13, G23]		S12 [S_{13}, S_{23}]	$\varepsilon 12$ [ε_{13}, ε_{23}]

Note: [ ] Through-thickness properties

Table 1 Elastic and strength property data for orthotropic materials

specimen are reinforced with adhesively bonded end tabs made from a glass-fibre-reinforced cross-ply or fabric laminate with the fibre axes of the fabric set at ±45° to the specimen axis, with a 90° tab angle (i.e. not tapered).

Longitudinal and transverse strain can be measured using strain gauges, extensometers or non-contact techniques. Bending strain should not exceed 3% of the overall strain. Testing is relatively straightforward using either a standard mechanical test frame or servo-hydraulic test machine. Tensile tests using straight edge specimens, as specified above, have proved satisfactory for determining the long-term performance of composite laminates exposed to a combination of stress (static and cyclic fatigue), thermal and chemical environments. Strain is relatively uniform within the specimen gauge-length. ISO 13003 specifies test methods for determining fatigue properties (including tension–tension) of composite laminates under cyclic loading conditions.

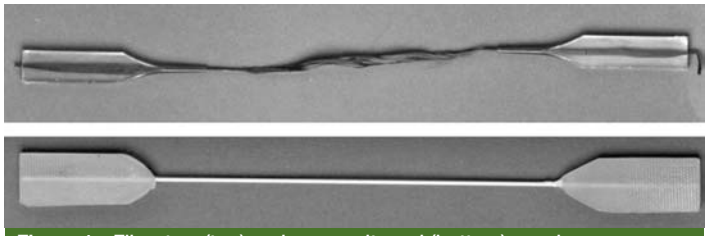

Figure 1 Fibre tow (top) and composite rod (bottom) specimens

Figure 2 Long-term environmental static fatigue apparatus

Through-thickness tension

There are two contrasting methods and several associated geometries, which can be employed in the measurement of T-T tension (i.e. direct and indirect tensile loading) – see Tarnopol'skii and Kincis, 1985; Broughton *et al.*, 1994, 1999; and Hodgkinson, 2000. The direct method introduces tensile load to parallel-sided (square cross-section) or waisted short block specimens via adhesively bonded metallic (reusable) loading bars (see **Figure 4**). The indirect method aims to induce T-T tension in significantly curved specimens by the application of bending moments. Indirect methods (e.g. C-section) tend to produce mixed-mode failure and not T-T tension.

Figure 3 Unidirectional (top) and multidirectional (bottom) tensile specimens

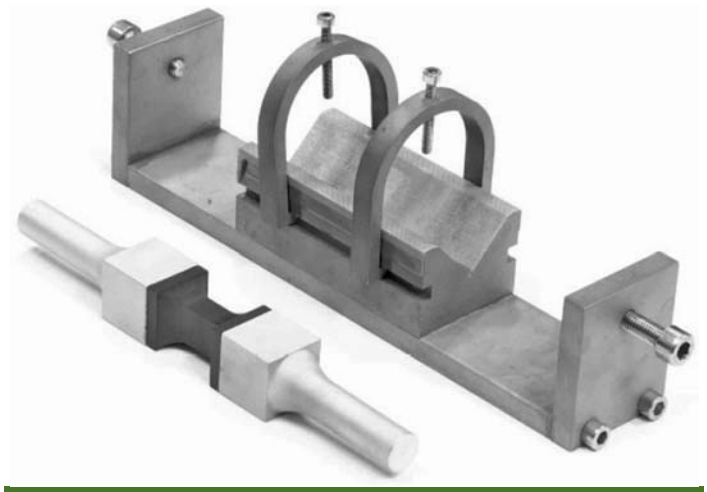

Figure 4 RARDE T-T tension specimen with bonding fixture

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Short parallel-sided blocks (~40 mm high (laminate thickness) and 15 mm square) used to measure T-T elastic properties are unsuitable for measuring T-T tensile strength. Biaxial strain gauges (2 or 3 mm gauge length) are bonded onto the mid-line of each of the four sides of the specimen to measure axial and transverse strains. This enables average strains to be calculated, thus accounting for bending due to small deviations in specimen or load alignment. Strength results are particularly sensitive to system alignment and load eccentricity. All faces need to be flat and parallel (to within ± 0.1 mm). A special bonding fixture (**Figure 4**) is required to ensure good alignment and to maintain pressure on the bonding surfaces during cure.

Elliptical, or circular waisted block specimens (**Figure 5**), such as those specified in ASTM D 7291 and NPL draft procedure 'Fibre-reinforced Plastic Composites – Determination of Through-thickness Tensile Properties of Fibre-reinforced Plastic Composites' can be used to determine T-T tensile strength. The reduction in cross-sectional area (CSA) promotes failure at the specimen mid-thickness with failure occurring in a plane normal to the applied load. The tensile strength is simply the applied load at failure divided by the cross-sectional area at the specimen mid-section. Using specimens with large circular radius or elliptical fillet reduces the stress concentration in the vicinity of the fillet root. Thinner material can be tested provided the laminate thickness is greater than 20 mm and linear dimensions of the specimen are scaled in proportion to the standard geometry. The inclusion of a rectangular gauge-section (e.g. RARDE specimen (**Figure 4**)) enables both strength and elastic properties to be obtained using the same specimen.

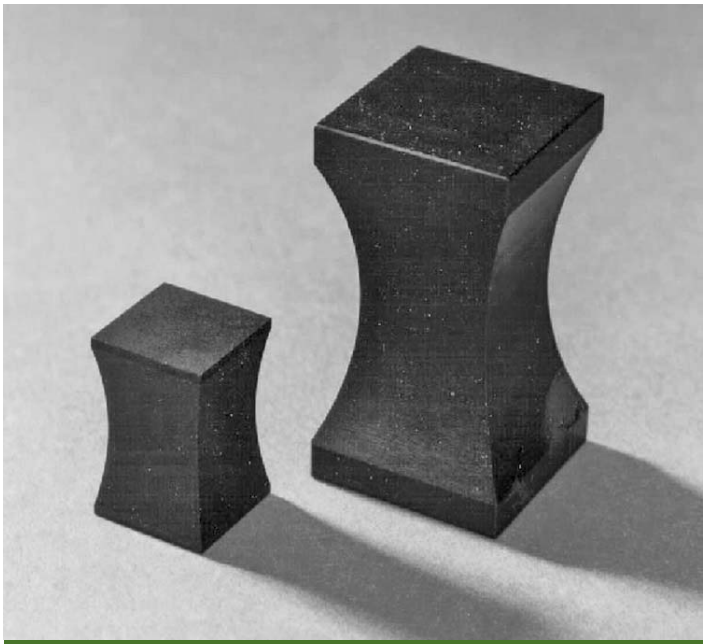

Figure 5 Waisted block T-T specimens

Compression

Polymers

Compression properties of polymers can be obtained by loading small rectangular specimens between two parallel, hardened stainless steel platens in accordance with ISO 604. The recommended specimen length is 50 mm and 10 mm for modulus and strength measurement, respectively. Both specimen types are 10 mm wide and have a thickness of 4 mm. The compressive strength is simply the applied load at failure divided by the CSA of the gauge section (i.e. $\sigma = P/A$). Provided the specimen does not bend and/or buckle under compressive loads, the data analysis is relatively straightforward. Extensometers or strain gauges are used to measure longitudinal and lateral strains/displacements. For maximum accuracy, strain needs to be measured on the two opposing faces of rectangular specimens. Failure can be expected to initiate at the specimen ends of thermoset resins due to stress concentrations; however, thermoplastics tend to undergo shear failure.

Composites

Compression standard ISO 14126 allows for end, shear and combined loading. It is important when using any of the acceptable test fixtures specified in the standard (i.e. Celanese, IITRI (Illinois Institute of Technology Research Institute) and end-loading blocks) to ensure good axial alignment. Hydraulic grips in aligned test machines are also acceptable. In all cases, the gauge length is unsupported. ISO 14126 recommends using specimens 110 mm long, 10 mm wide with a gauge length of 10 mm. The required thickness is 2 mm for continuous aligned materials and between 2 and 10 mm for multidirectional laminates and random fibre reinforcements. The standard requires strain gauges on both specimen faces and for the test to be completed, or terminated, within a bending (buckling) strain of less than 10% of the axial strain.

Specimens are end tabbed to prevent failure at the loaded ends of the specimen, although for random fibre formats this is not always necessary. The end-loading fixture shown in **Figure 6** (Shaw and Sims, 2005) can accommodate a broad range of specimen thicknesses, making it more suitable for thick section compression testing than the IITRI or Celanese rigs, which require specifically made wedges to adapt to each different thickness. A four-pillar die set is used with the end-loading fixture to maintain uniform compression loading.

Through-thickness compression

The direct tensile specimen geometries described previously (see also NPL draft procedure 'Fibre-reinforced Plastic

Figure 6 End-loading compression fixture with specimen

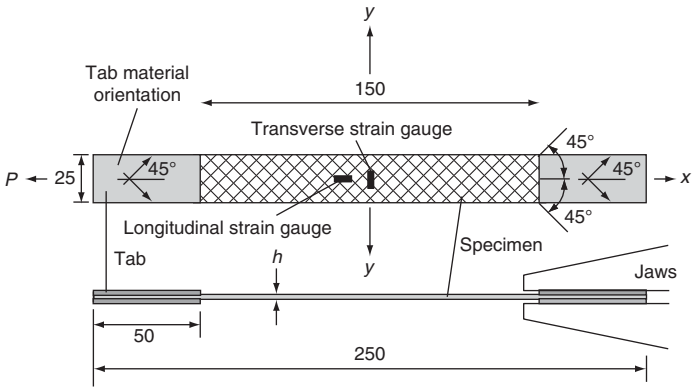

Figure 8 Schematic of the ±45° tension specimen

Composites – Determination of Through-thickness Compressive Properties of Fibre-reinforced Plastic Composites') can also be used to determine T-T compression properties. Specimens are loaded in compression between flat, parallel, hardened stainless steel platens with recesses to reduce lateral movement of the specimen. A four-pillar die set is used to maintain uniform compression loading (**Figure 7**). Shear is the predominant cause of failure in all cases, independent of material microstructure, loading configuration or specimen size (Lodeiro *et al.*, 1999).

Shear

Commonly used methods for the determination of shear properties include uniaxial tension of a balanced symmetric ±45° laminate (ISO 14129 and ASTM D 3518), plate-twist (ISO 15310) and V-notched beam (ASTM D 5379) shear tests. 10° off-axis, two-rail and three-rail shear, and torsion of thin-walled tube test geometries are also used to a lesser degree – see Hodgkinson, 2000. The plate twist, V-notched beam and torsion rod (ASTM D 1043) tests can be used to provide shear property data for polymers. The ±45° tension test (**Figure 8**) can be used to determine in-plane shear properties of continuous aligned fibre-reinforced systems. Test geometry and loading configuration is similar to that employed for tensile loading of the multidirectional laminates (250 mm × 25 mm × 2 mm). It is recommended that for materials constructed with layers (plies) thicker than 0.125 mm, the laminate should consist of 16 layers (i.e. $[+45°/-45°]_{4S}$).

Figure 7 RARDE compression specimen and loading fixture

The average shear stress is the applied load divided by twice the cross-sectional area. The test is terminated at or before 5% shear strain, thus shortening the test duration, which can be excessive for tough matrix systems. The applied stress at failure or 5% strain equates to shear strength. The 5% shear strain limit also minimises fibre rotation (scissoring) and internal heating effects generated due to friction. Longitudinal and transverse strains, which can be measured using either strain gauges or extensometers, are required for determining the shear modulus.

Plate-twist method can be used to measure shear modulus ranging from 0.29 to 88.2 GPa. It is unsuitable for determining shear strength. The test consists of a square plate (150 × 150 mm) supported on the two corners of one diagonal with load applied to the corners of the opposite diagonal (**Figure 9**). The plate length-to-thickness h ratio should be 35 to minimise T-T shear effects. V-notched beam method can be used to measure shear modulus and shear strength in all of the three shear planes (1–2, 1–3 and 2–3). The test employs a double edge-notched, flat rectangular specimen (76 mm × 20 mm × 5 mm) – see **Figure 9**. Two 90° angle notches with a notch root radius of 1.3 mm are machined at the specimen mid-length of each with faces oriented at ±45° to the longitudinal axis, to a depth of 20% of the specimen width (i.e. 4 mm).

The average shear stress is the applied load divided by the cross-sectional area between the notches. Shear strain is measured using biaxial strain gauges (1 or 2 mm gauge length) aligned ±45° to the longitudinal axis bonded to both faces of the specimen. Shear modulus is determined from the average response of the back-to-back biaxial rosettes. A special test fixture is required.

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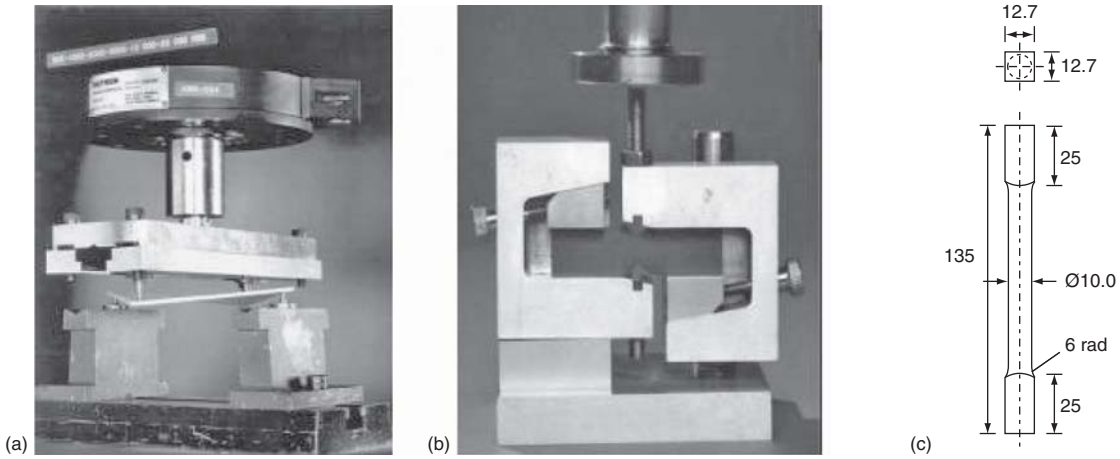

Figure 9 Shear methods for testing polymers: (a) plate twist; (b) V-notched beam; (c) torsion rod

Through-thickness shear

The short beam and double-notch shear tests can be used to measure interlaminar shear strength. The short beam shear test (ISO 14130) can be used to test continuous aligned and 2D woven fabric laminates, provided an interlaminar shear failure is obtained. Most other materials tend to fail in tension (bottom surface) or from a combination of high shear and transfibrile compression (top surface). The test consists of a short rectangular beam loaded in three-point bending. The standard specimen thickness h is 2 mm (nominally) with a width and overall length of 10 and 20 mm, respectively. The recommended width is $5h$ and overall length is $10h$ for material sectioned from panels with a thickness in excess of 2 mm. In all cases, the loading span is $5h \pm 0.3$ mm.

The double-notch shear test consists of loading an asymmetrically notched composite specimen in either tension (BS 4994 and BS 6464) or compression (ASTM D 3846 (or NPL draft procedure 'Fibre-reinforced Plastic Composites – Determination of Through-thickness Shear Strength Using the Double-notch Shear Test')). The tension specimens are unsupported, whereas compression specimens (**Figure 10**) are end-loaded and supported along the entire length of the specimen to minimise out-of-plane deformation (see ASTM D 695). Shear strength is the applied load at failure divided by the area between the notches. The double-notch shear method provides consistent strength data (low scatter) with interlaminar failure occurring along the mid-plane joining the notches (Broughton *et al.*, 2001; Shaw and Sims, 2003).

Flexure

Flexure tests, which are routinely employed throughout the plastics and composites industry for quality assurance and material selection purposes, are not suitable for generating engineering data as the tests are structural. Specimen

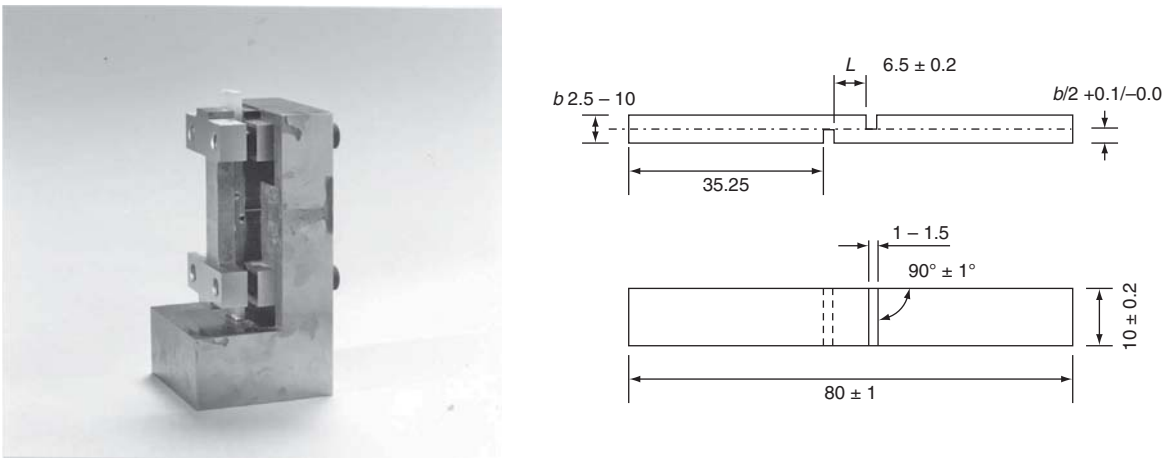

Figure 10 Compression fixture with double-notch shear specimen

preparation and testing is relatively straightforward, fast and economic with data reduction posing no particular problems. Testing is carried out using a special loading fixture, which is attached to the loading frame. Commercial fixtures are available at a moderate cost. Displacement is measured directly from the test machine cross-head or by using a linear variable displacement transducer (LVDT). ISO 14125 includes three-point and four-point bending configurations. Advantages of using a four-point bending arrangement are that the bending moment between the central loading points is uniform and the loads applied to the inner loading points are halved. Flexure tests have been readily adapted to fatigue, creep and environmental testing. ISO 13003 specifies a method for flexural fatigue of composites by constant-amplitude loading.

Fracture toughness

Delaminations are probably the most life-limiting defects that occur in layered or laminated structures, and may arise during processing or subsequent service. Crack initiation and growth usually occurs under mixed-mode conditions, a combination of Mode I (crack-opening), Mode II (forward-shear) and Mode III (scissor-shear or tear). As a result, fracture toughness tests have been developed to quantify resistance to crack initiation and growth under tensile and shear loading conditions. Research in recent years has been directed towards generating reliable test methods for determining fracture toughness under Mode I (ISO 15024 (static) and ASTM D 5528 (static), ASTM D 6115 (fatigue)), Mode II and Mixed-Mode I/II (ASTM D 6671) loading conditions, and to a lesser degree Mode III (see Friedrich, 1989).

Mode I – double cantilever beam (DCB)

The test is used to measure the initiation and propagation values of fracture toughness G_I under static and cyclic loading conditions. A tensile load is applied to a specimen with an embedded through-width insert (i.e. debond) at the specimen mid-plane (**Figure 11**). The tensile force acts in a direction normal to the crack surface. DCB specimens are 125 mm long, 20 mm wide with a thickness of 3 mm and 5 mm for carbon and glass-fibre-reinforced systems, respectively. The insert length is 60 mm. Crack length is measured using either a travelling microscope, a crack gauge or video camera.

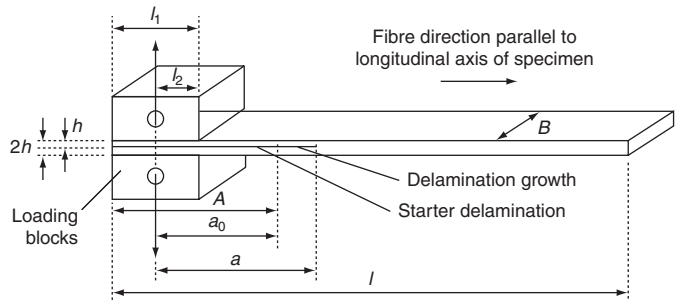

Figure 11 DCB test specimen

Mode II

There are several potential test methods including the end notched flexure (ENF) and end loaded split (ELS) methods. The ENF test has emerged as the most commonly used test method for quantifying G_{IIc} for metallic and laminated polymer composite adherends. The ENF test is essentially a DCB specimen loaded in either three-point or four-point flexure. The dimensions are similar to those employed for the DCB specimen. The test is straightforward and can be conducted using standard mechanical test equipment and a suitable flexure fixture. Analysis is also straightforward, although there is concern as to the method of pre-cracking specimens.

Mixed-mode I/II

The work on mixed-mode I/II test geometries, such as the mixed-mode bend (MMB) and crack lap shear (CLS) can be considered still in its infancy. The MMB test consists of clamping one end of a DCB specimen and applying load to one of the adherends. By using adherends of different bending stiffness (i.e. altering the thickness ratio of the two adherends) the ratio of G_I/G_{II} can be varied. Test specimen dimensions and fabrication are similar to those used for DCB and ENF specimens. A special fixture is required. The CLS method enables the magnitude of the shear and peel stress components to be varied, by changing the relative thickness of adherends. The specimen can be loaded in either tension or flexure (four-point bend). Failure is Mode II dominated with G_I/G_{II} ratios ranging from 20 to 35%. The Mode I component increases when the CLS specimen is subjected to pure bending, also the energy release rate is independent of crack length.

Quality assessment

It is equally important in any testing programme that the measured data are accurate and the material is accurately defined in order to ensure its maximum value and relevance (i.e. the material is fully and correctly characterised). This particularly applies to composite material as the material is generally only formed with its intended properties if the fabrication process including the cure and any post-cure are correctly undertaken. Properties that should be measured are the fibre and void fractions (ISO 14127 for carbon-fibre based systems, ISO 1172 and 7822 for glass-fibre based systems), glass-transition temperature as an indication of degree of cure (ISO 6721-11 for DMA measurements, ISO 11357 for DSC measurements) and an assessment made of ply directions when relevant.

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Future trends

A number of new initiatives are under way to provide new and improved test methods for characterisation of elastic properties, interfacial (and interphase) properties, scaling effects (i.e. thick-sections), open-hole (notched) tension and compression strengths, and material performance under multi-axial loading conditions. Ultrasonic techniques (e.g. time-of-flight) offer an alternative to conventional mechanical testing for the determination of elastic properties, state of cure or level of degradation, although the use of ultrasonic techniques for characterisation of composite materials has been limited (c.f. use for NDT inspection). Many of the difficulties encountered in mechanical testing, particularly for evaluating T-T properties, are obviated by the ultrasonic pulse method since the applied stress is negligibly small, and a high sensitivity can be obtained on small, thin specimens.

Novel measurement techniques, such as nanoindentation, nanoscratch and atomic force microscopy (AFM), and micromechanical interface test methods (e.g. single-fibre fragmentation) are being assessed for the purpose of characterising fibre-matrix interfacial properties. These methods potentially offer rapid and/or low-cost alternatives to mechanical testing for quality assurance (production) and environmental degradation assessment. No single ideal or universally applicable interfacial measurement method exists that can provide quantitative data for engineering or design purposes.

The general perception has been to associate T-T properties with thick composites, whereas in fact an equally important issue relates to the measurement of in-plane properties and the effect of physical size of test specimens on measured data. Although extensive developmental work has been undertaken worldwide into test methods and design procedures for in-plane properties of thin laminates, there are no standard test methods available that provide guidance on testing of thick composite sections. The approach generally adopted has been to use existing standards, developed for testing small laboratory-scale specimens, with non-standard (i.e. larger) specimen geometries. A major concern relates to whether data generated from standards for relatively thin specimens are equivalent to or representative of thick-section mechanical behaviour.

Progress has been made in recent years to standardise test methods for determining pin-bearing strength, and open-hole (notched) and filled-hole tensile and compressive strengths. ASTM standards are now available for measuring these properties. A parallel exercise is now under way to provide BSI and ISO standards for both static and cyclic loading conditions. Multi-axial testing, which can be used to identify actual material strengths and failure mechanisms under representative service conditions, is often used simply on the grounds that there is insufficient confidence in 3D failure analysis. Testing consists of applying loads/displacements along the primary, mutually orthogonal coordinate axes of lineal test specimens (i.e. rectangular plates, cubes or 2D or 3D cruciforms – see **Figure 12**) or, alternatively, loading cylindrical specimens either in tension, compression and/or shear. Test methods/standards are required for multi-axial loading of lineal and thick-walled cylindrical specimens.

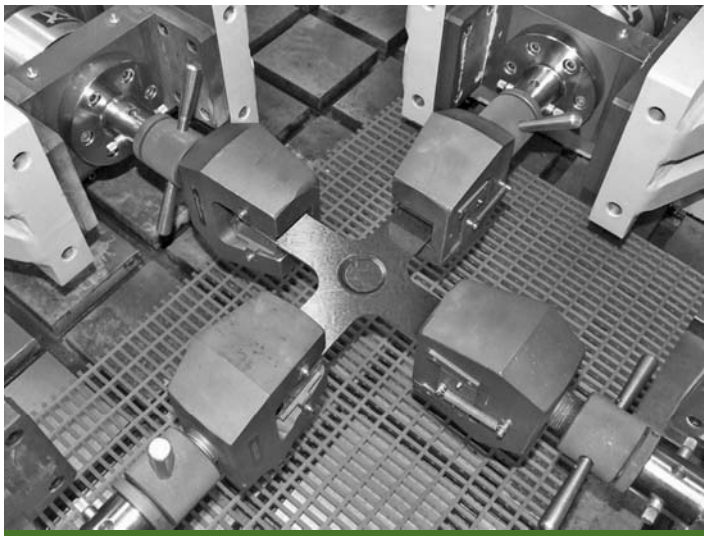

Figure 12 Biaxial testing of cruciform specimen

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Standards

ASTM D 695: *Standard Test Method for Compressive Properties of Rigid Plastics*.
ASTM D 1043: *Standard Test Method for Stiffness Properties of Plastics as a Function of Temperature by Means of a Torsion Test*.
ASTM D 3518: *Standard Test Method for In-plane Shear Response of Polymer Matrix Composite Materials by Tensile Test of a* $\pm 45^\circ$ *Laminate*.
ASTM D 3846: *Standard Test Method for In-plane Shear Strength of Reinforced Plastics*.
ASTM D 4018: *Standard Test Methods for Properties of Continuous Filament Carbon and Graphite Fiber Tows*.
ASTM D 5379: *Standard Test Method for Shear Properties of Composite Materials by the V-notched Beam Method*.
ASTM D 5528: *Standard Test Method for Mode I Interlaminar Fracture Toughness of Unidirectional Fiber-reinforced Polymer Matrix Composites*.
ASTM D 6115: *Standard Test Method for Mode I Fatigue Delamination Growth Onset of Unidirectional Fiber-reinforced Polymer Matrix Composites*.
ASTM D 6671: *Standard Test Method for Mixed Mode I–Mode II Interlaminar Fracture Toughness of Unidirectional Fiber Reinforced Polymer Matrix Composites*.
ASTM D 7291: *Standard Test Method for Through-thickness 'Flatwise' Tensile Strength and Elastic Modulus of a Fiber-reinforced Polymer Matrix Composite Material*.
ISO 527-2: *Plastics – Determination of Tensile Properties – Part 2: Test Conditions for Moulding and Extrusion Plastics*.
ISO 527-4: *Plastics – Determination of Tensile Properties – Part 4: Test Conditions for Isotropic and Orthotropic Fibre-reinforced Plastic Composites*.
ISO 527-5: *Plastics – Determination of Tensile Properties – Part 5: Test Conditions for Unidirectional Fibre-reinforced Plastic Composites*.
ISO 604: *Plastics – Determination of Compressive Properties*.
ISO 1172: *Textile-glass-reinforced Plastics – Prepregs, Moulding Compounds and Laminates – Determination of the Textile-glass and Mineral-filler Content – Calcination Methods*.
ISO 3341: *Textile Glass. Yarns – Determination of Breaking Force and Breaking Elongation*.
ISO 6721-11: *Plastics – Determination of Dynamic Mechanical Properties – Part 11: Glass Transition Temperature*.
ISO 7822: *Textile Glass Reinforced Plastics – Determination of Void Content – Loss on Ignition, Mechanical Disintegration and Statistical Counting Methods*.
ISO 9163: *Textile Glass – Rovings – Manufacture of Test Specimens and Determination of Tensile Strength of Impregnated Rovings*.
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ISO 14125: *Fibre-reinforced Plastic Composites – Determination of Flexural Properties*.
ISO 14126: *Fibre-reinforced Plastic Composites – Determination of Compressive Properties in the In-plane Direction*.
ISO 14127: *Carbon-fibre-reinforced Composites – Determination of the Resin, Fibre and Void Contents*.
ISO 14129: *Fibre-reinforced Plastic Composites – Determination of the In-plane Shear Stress/Shear Strain Response, Including the In-Plane Shear Modulus and Strength, by the Plus or Minus 45 Degree Tension Test Method*.
ISO 14130: *Fibre-reinforced Plastic Composites – Determination of Apparent Interlaminar Shear Strength by Short-Beam Method*.
ISO 15024: *Fibre-reinforced Plastic Composites – Determination of Mode I Interlaminar Fracture Toughness,* G_{IC}*, for Unidirectionally Reinforced Materials*.
ISO 15310: *Reinforced Plastics – Determination of the In-plane Shear Modulus by the Plate Twist Method*.

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Chapter 54

Mechanical properties of FRP composites

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This chapter describes a two-step analysis procedure commonly used for the prediction of the mechanical properties of a laminated fibre-reinforced polymers (FRP) composite from the properties of the two constituent materials (i.e. the fibres and the matrix) as well as the structure of the composite (i.e. the directions and stacking sequence of the fibres). In this analysis procedure, the first step is to determine the properties of a unidirectional FRP lamina using a micromechanics approach while the second step is to determine the properties of an FRP laminate using the classical lamination theory.

Introduction

Fibre-reinforced polymer (FRP) composites are commonly formed by embedding continuous fibres in a polymeric resin matrix. The mechanical properties of an FRP composite depend not only on the properties of the two constituent materials (i.e. the fibres and the matrix), but also on the directions and volume fraction of the fibres, as discussed in Chapter 52. This characteristic of FRP composites offers the possibility for designing and optimising the structure of the composite material to arrive at desirable properties for specific applications. This chapter describes a two-step analysis procedure commonly used for the prediction of the macroscopic mechanical properties of an FRP composite from the properties of the two constituent materials (i.e. the fibres and the matrix) and how they are arranged in forming the composite.

The content of the chapter is limited to laminated FRP composites (or FRP laminates) which are formed from two or more unidirectional laminae stacked together (**Figure 1**), as almost all FRP products used in the construction industry are made of FRP laminates. The general concepts presented in this chapter are however generally extendable to other FRP composites.

A unidirectional lamina consists of unidirectional fibres embedded in a matrix. In a laminate, the fibre directions of different laminae are typically different. At the macroscopic level, an FRP laminate (or a unidirectional FRP lamina) can be regarded as a quasi-homogeneous material with uniform mechanical properties everywhere. However, the laminate is heterogeneous at the microscopic level, being a combination of two constituent materials. For structural applications in civil engineering, one is mainly interested in the macroscopic mechanical properties of an FRP composite. The two-step analysis procedure presented in this chapter is for the prediction of these macroscopic mechanical properties and includes the following two steps: (i) micromechanical analysis for predicting the mechanical properties of a lamina from those of the two constituent materials and their volume fractions; and (ii) macromechanical analysis for predicting the mechanical properties of the laminate from those of the laminae and their stacking sequence, using the classical lamination theory. The section on Micromechanics of a lamina deals with the first step of the analysis procedure, followed by a discussion of the stress–strain relations of a lamina in the section on Stress–strain relationship of a lamina. The second step of the analysis procedure is described in the section on Classical lamination theory. In the section on Additional information, the limitations of the analysis procedure and additional information relevant to the topics of the chapter are briefly discussed.

Micromechanics of a lamina

An FRP lamina is heterogeneous microscopically because the two constituents (i.e. the fibres and the matrix) have different properties. Macroscopically, a unidirectional FRP lamina can usually be treated as an orthotropic material whose mechanical properties in the fibre direction are different from those in the other two orthogonal directions. That is, the elastic modulus, shear modulus and Poisson's ratios are different in different directions.

For ease of reference, the fibre direction, the transverse direction in the lamina plane and the through-thickness direction are termed respectively the longitudinal (axis-1), transverse (axis-2) and through-thickness (axis-3) directions hereafter. For simplicity, this chapter only deals with the mechanical properties in the lamina (longitudinal-transverse) plane. The mechanical properties in the plane of the longitudinal and through-thickness axes are usually the same as those in the longitudinal-transverse plane.

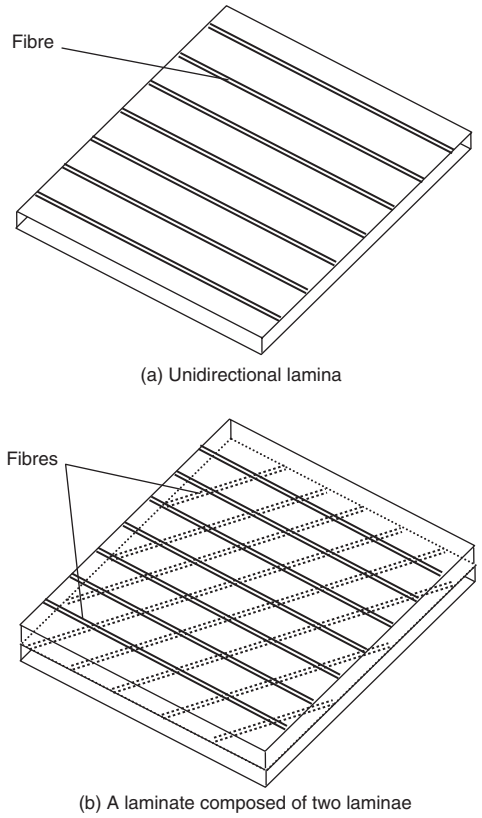

Figure 1 Unidirectional lamina and laminate

The sign convention for stresses and strains is that conventionally used in the theory of elasticity.

The mechanical properties of an FRP lamina may be obtained from those of the two constituents and their volume fractions based on micromechanics approaches which consider the local states of stresses and deformation of the two constituents and their interaction based on various simplifying assumptions (Jones, 1999; Daniel and Ishai, 2006). Many different micromechanics approaches are available and may generally be classified into the following approaches (Chamis and Sendeckyj, 1968; Whitney and McCullough, 1990):

1 mechanics of materials approach
2 semi-empirical approach
3 self-consistent field approach
4 variational approach
5 classical elasticity theory; and
6 numerical approach.

The first two approaches are simple in form and are widely accepted. They are described below.

Mechanics of materials approach

Assuming that the fibres and the matrix deform compatibly so that they have the same strain in the fibre (longitudinal) direction when the lamina is subjected to uniaxial loading in that direction, the longitudinal modulus E_1 and the major Poisson's ratio ν_{12} of a lamina can be found from the following simple equations known as the rule of mixtures:

$$E_1 = E_{f1}V_f + E_mV_m \tag{1a}$$

$$\nu_{12} = \nu_{f1}V_f + \nu_mV_m \tag{1b}$$

in which E_{f1} is the elastic modulus of the fibres in their longitudinal direction; ν_{f1} is the major Poisson's ratio of fibres; E_m and ν_m are the elastic modulus and the Poisson's ratio of the matrix respectively; and V_f and V_m are the volume fractions of fibres and matrix respectively. Note that the major Poisson's ratio ν_{12} is defined as the absolute value of the ratio of the transverse strain to the longitudinal strain when a lamina is under uniaxial loading in the longitudinal direction.

Based on the assumption that the transverse normal stress in the fibres is the same as that in the matrix when the lamina is subjected to transverse uniaxial loading, the transverse elastic modulus E_2 of a lamina can be found as

$$E_2 = \frac{E_{f2}E_m}{E_{f2}V_m + E_mV_f} \tag{2}$$

in which E_{f2} is the elastic modulus of fibres in the direction transverse to the fibres.

Similarly, assuming that the shear stress in the fibres is the same as that in the matrix, the in-plane shear modulus G_{12} of a lamina is given by

$$G_{12} = \frac{G_{f12}G_m}{G_{f12}V_m + G_mV_f} \tag{3}$$

where G_{f12} is the shear modulus of fibres and G_m is the shear modulus of matrix.

Equation (1a) represents a linear relationship between the longitudinal modulus of a lamina and the volume fraction of the fibres. As the fibres are usually much stiffer than the matrix, E_1 is a fibre-dominant property. Conversely, Equations (2) and (3) imply that E_2 and G_{12} are matrix-dominant properties, on which the properties of the fibres do not have a significant effect except when the fibre volume fraction is close to unity. The Poisson's ratios of fibres and matrix are typically similar, so the Poisson's ratios of a composite are similar to those of the two constituents.

Experimental studies have shown that the predictions of Equation (1a) are generally in good agreement with test results, but Equations (2) and (3) typically underestimate the actual values due to inaccuracy arising from the simplifying assumptions. For example, the assumption of equal transverse stresses adopted in deriving Equation (2) leads to a mismatch in the transverse strain at the interface between a fibre and the matrix. Indeed, research has shown that when a lamina is subjected to transverse loading, the stress of the matrix in the vicinity of fibres is

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highly non-uniform due to its interaction with the fibres (Jones, 1999; Daniel and Ishai, 2006). In deriving the above equations, it is also assumed that the fibres are uniformly distributed in the matrix while in practice the fibres are often randomly distributed with some in contact and others not (Jones, 1999). To account for the effects of these issues, more complicated approaches are needed, which are briefly discussed later in this section.

The Halpin–Tsai equations

Halpin and Tsai (1967) developed a semi-empirical approach which employs an interpolation procedure to predict the mechanical properties, especially E_2 and G_{12}, of a lamina. The Halpin–Tsai equation for the transverse modulus is

$$E_2 = \frac{E_m(1 + \xi_E \eta V_f)}{1 - \eta V_f} \tag{4}$$

where

$$\eta = \frac{E_{f2} - E_m}{E_{f2} + \xi_E E_m} \tag{5}$$

and ξ_E is a parameter to be determined from experiments.

Similarly, the equation for the in-plane shear modulus is

$$G_{12} = \frac{G_m(1 + \xi_G \eta V_f)}{1 - \eta V_f} \tag{6}$$

where

$$\eta = \frac{G_{f12} - G_m}{G_{f12} + \xi_G G_m} \tag{7}$$

and ξ_G is again a parameter to be determined from experiments. When $\xi_E = \xi_G = 0$, Equations (4) and (6) reduce to Equations (2) and (3) respectively. When $\xi_E = \xi_G = \infty$, Equations (4) and (6) reduce to the following well-known rule of mixtures:

$$E_2 = E_{f2} V_f + E_m V_m \tag{8a}$$

$$G_{12} = G_{f12} V_f + G_m V_m \tag{8b}$$

which are similar in form to Equations (1a) and (1b).

Experimental studies have shown that ξ_E and ξ_G are affected by many factors including the type of cross-sectional arrays (e.g. square array, hexagonal array), the shape, and the volume fraction of fibres (Jones, 1999; Daniel and Ishai, 2006). For circular fibres in a square array, usually $\xi_E = 2$ and $\xi_G = 1$.

Other approaches

In order to improve the accuracy of prediction for the properties of a lamina, many other approaches have been developed, as mentioned earlier. They generally consider the geometric (e.g. fibre distribution) and physical (e.g. deformation compatibility) conditions in a more rigorous manner. However, they typically lead to solutions which are either more complicated or not in closed-form (e.g. numerical approaches). Consequently, these approaches are not suitable for practical application in civil engineering design and not discussed in detail herein. Further information on these approaches can be found in Chamis and Sendeckyj (1968) and Whitney and McCullough (1990).

Stress–strain relationship of a lamina

As mentioned earlier, a unidirectional FRP lamina can be treated as a quasi-homogeneous orthotropic material at the macroscopic level. Its mechanical properties vary with direction but it has three mutually perpendicular planes of material symmetry. The intersections of these planes define three principal material axes, as discussed in the section Mechanics of materials approach. These three directions represent the principal directions of orthotropy. As the composing laminae of an FRP laminate are subjected mainly to in-plane loading in most civil engineering applications, this section is again concerned only with their in-plane constitutive behaviour. The in-plane stress–strain relationship of a lamina in the principal directions of orthotropy is first presented, followed by an examination of the transformation of stresses and strains into arbitrary directions.

Stress–strain relationship in the principal directions of orthotropy

The in-plane stress–strain relationship of an FRP lamina can be represented by the following equation, where $[Q]$ is the stiffness matrix:

$$\begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{bmatrix} = [Q] \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{bmatrix} = \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ Q_{21} & Q_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{bmatrix} \tag{9}$$

in which

$$Q_{11} = \frac{E_1}{1 - \nu_{12}\nu_{21}} \tag{10}$$

$$Q_{22} = \frac{E_2}{1 - \nu_{12}\nu_{21}} \tag{11}$$

$$Q_{12} = Q_{21} = \frac{\nu_{12} E_2}{1 - \nu_{12}\nu_{21}} = \frac{\nu_{21} E_1}{1 - \nu_{12}\nu_{21}} \tag{12}$$

$$Q_{66} = G_{12} \tag{13}$$

where σ_1 and σ_2 are the normal stresses in the longitudinal direction and the transverse direction respectively; ε_1 and ε_2 are the corresponding normal strains respectively; τ_{12} and γ_{12} are the in-plane shear stress and shear strain respectively; ν_{21} is the transverse Poisson's ratio (i.e. ratio of longitudinal strain to transverse strain when a lamina is subjected to loading in the transverse direction). Because

of the symmetry of the stiffness matrix, only three of the four elastic constants E_1, E_2, ν_{12} and ν_{21} are independent, and they are related by the following equation:

$$\frac{\nu_{12}}{E_1} = \frac{\nu_{21}}{E_2} \tag{14}$$

Equation (9) can be inverted to give the following strain–stress relationship where $[S]$ is the compliance matrix:

$$\begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{bmatrix} = [S] \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{bmatrix} = \begin{bmatrix} S_{11} & S_{12} & 0 \\ S_{21} & S_{22} & 0 \\ 0 & 0 & S_{66} \end{bmatrix} \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{bmatrix} \tag{15}$$

where

$$S_{11} = \frac{1}{E_1} \tag{16}$$

$$S_{22} = \frac{1}{E_2} \tag{17}$$

$$S_{12} = S_{21} = -\frac{\nu_{12}}{E_1} = -\frac{\nu_{21}}{E_2} \tag{18}$$

$$S_{66} = \frac{1}{G_{12}} \tag{19}$$

Transformation of stresses and strains

When a lamina is loaded in an arbitrary direction and/or when the reference coordinate system does not coincide with the principal material coordinate system, transformation of stresses and strains is usually required. The stresses and strains in the principal material coordinate system (1–2) can be transformed to those in an arbitrary coordinate system (x–y) through the following equations:

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \begin{bmatrix} \cos^2\theta & \sin^2\theta & -2\sin\theta\cos\theta \\ \sin^2\theta & \cos^2\theta & 2\sin\theta\cos\theta \\ \sin\theta\cos\theta & -\sin\theta\cos\theta & \cos^2\theta - \sin^2\theta \end{bmatrix} \times \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{Bmatrix} \tag{20}$$

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy}/2 \end{Bmatrix} = \begin{bmatrix} \cos^2\theta & \sin^2\theta & -2\sin\theta\cos\theta \\ \sin^2\theta & \cos^2\theta & 2\sin\theta\cos\theta \\ \sin\theta\cos\theta & -\sin\theta\cos\theta & \cos^2\theta - \sin^2\theta \end{bmatrix} \times \begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12}/2 \end{Bmatrix} \tag{21}$$

where θ (see **Figure 2**) is the angle measured counterclockwise from the x-axis to the 1-axis (i.e. the fibre direction).

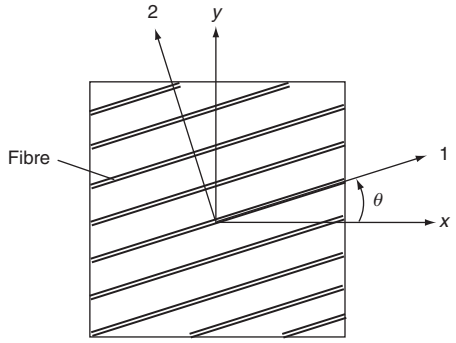

Figure 2 Reference coordinate system and principal material coordinate system

With Equations (9), (20) and (21), the stress–strain relationship of an FRP lamina in an arbitrary coordinate system can be expressed as

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = [\overline{Q}] \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \overline{Q_{11}} & \overline{Q_{12}} & \overline{Q_{16}} \\ \overline{Q_{12}} & \overline{Q_{22}} & \overline{Q_{26}} \\ \overline{Q_{16}} & \overline{Q_{26}} & \overline{Q_{66}} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \tag{22}$$

where

$$\overline{Q_{11}} = Q_{11}\cos^4\theta + 2(Q_{12} + 2Q_{66})\sin^2\theta\cos^2\theta + Q_{22}\sin^4\theta \tag{23}$$

$$\overline{Q_{12}} = (Q_{11} + Q_{22} - 4Q_{66})\sin^2\theta\cos^2\theta + Q_{12}(\sin^4\theta + \cos^4\theta) \tag{24}$$

$$\overline{Q_{22}} = Q_{11}\sin^4\theta + 2(Q_{12} + 2Q_{66})\sin^2\theta\cos^2\theta + Q_{22}\cos^4\theta \tag{25}$$

$$\overline{Q_{16}} = (Q_{11} - Q_{12} - 2Q_{66})\sin\theta\cos^3\theta + (Q_{12} - Q_{22} + 2Q_{66})\sin^3\theta\cos\theta \tag{26}$$

$$\overline{Q_{26}} = (Q_{11} - Q_{12} - 2Q_{66})\sin^3\theta\cos\theta + (Q_{12} - Q_{22} + 2Q_{66})\sin\theta\cos^3\theta \tag{27}$$

$$\overline{Q_{66}} = (Q_{11} + Q_{22} - 2Q_{12} - 2Q_{66})\sin^2\theta\cos^2\theta + Q_{66}(\sin^4\theta + \cos^4\theta) \tag{28}$$

Similarly, Equation (22) can be inverted to the following strain–stress relationship in an arbitrary coordinate system:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = [\overline{S}] \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \begin{bmatrix} \overline{S_{11}} & \overline{S_{12}} & \overline{S_{16}} \\ \overline{S_{12}} & \overline{S_{22}} & \overline{S_{26}} \\ \overline{S_{16}} & \overline{S_{26}} & \overline{S_{66}} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \tag{29}$$

in which

$$\overline{S_{11}} = S_{11}\cos^4\theta + (2S_{12} + S_{66})\sin^2\theta\cos^2\theta + S_{22}\sin^4\theta \tag{30}$$

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$$\overline{S_{12}} = (S_{11} + S_{22} - S_{66})\sin^2\theta\cos^2\theta + S_{12}(\sin^4\theta + \cos^4\theta) \quad (31)$$

$$\overline{S_{22}} = S_{11}\sin^4\theta + (2S_{12} + S_{66})\sin^2\theta\cos^2\theta + S_{22}\cos^4\theta \quad (32)$$

$$\overline{S_{16}} = (2S_{11} - 2S_{12} - S_{66})\sin\theta\cos^3\theta + (2S_{12} - 2S_{22} + S_{66})\sin^3\theta\cos\theta \quad (33)$$

$$\overline{S_{26}} = (2S_{11} - 2S_{12} - S_{66})\sin^3\theta\cos\theta + (2S_{12} - 2S_{22} + S_{66})\sin\theta\cos^3\theta \quad (34)$$

$$\overline{S_{66}} = 2(2S_{11} + 2S_{22} - 4S_{12} - S_{66})\sin^2\theta\cos^2\theta + S_{66}(\sin^4\theta + \cos^4\theta) \quad (35)$$

Equations (22) and (29) show that, in an arbitrary coordinate system, both the stiffness matrix and the compliance matrix are fully populated with nine non-zero terms, which indicates that in an arbitrary coordinate system, normal stresses (strains) are coupled with shear strains (stresses). However, only four of the nine terms are independent. The nine terms of each matrix are implicitly related by Equations (23)–(28) (for the stiffness matrix) or Equations (30)–(35) (for the compliance matrix).

The elastic engineering constants in an arbitrary x–y coordinate system can be solved from the following equations:

$$\overline{S_{11}} = \frac{1}{E_x} \quad (36)$$

$$\overline{S_{22}} = \frac{1}{E_y} \quad (37)$$

$$\overline{S_{66}} = \frac{1}{G_{xy}} \quad (38)$$

$$\overline{S_{12}} = -\frac{\nu_{xy}}{E_x} = -\frac{\nu_{yx}}{E_y} \quad (39)$$

$$\overline{S_{16}} = \frac{\eta_{xs}}{E_x} = \frac{\eta_{sx}}{G_{xy}} \quad (40)$$

$$\overline{S_{26}} = \frac{\eta_{ys}}{E_y} = \frac{\eta_{sy}}{G_{xy}} \quad (41)$$

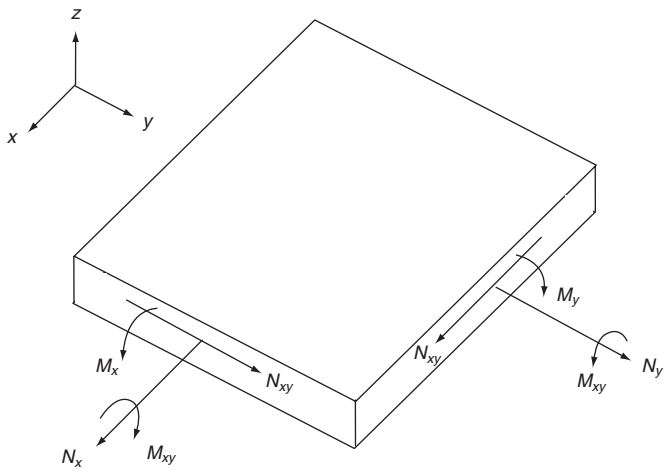
where E_x and E_y are the elastic moduli in the x- and y-directions respectively; G_{xy} is the shear modulus in the x–y plane; ν_{xy} and ν_{yx} are Poisson's ratios; η_{xs}, η_{sx}, η_{ys}, η_{sy} are known as the shear coupling coefficients; η_{sx} is the ratio of the normal strain in the x-direction to the shear strain and η_{sy} is the ratio of the normal strain in the y-direction to the shear strain, when the lamina is under pure shear loading in the x–y plane; η_{xs} is the ratio of the shear strain to the normal strain in the x-direction when the lamina is subjected to loading in the x-direction; η_{ys} is the ratio of the shear strain to the normal strain in the y-direction when the lamina is subjected to loading in the y-direction.

Classical lamination theory

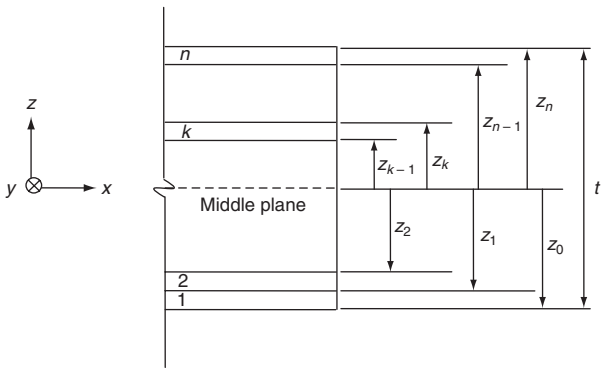
The mechanical properties of a multidirectional laminate may be calculated from those of the composing laminae and their stacking sequence using the classical lamination theory (CLT). The CLT is based on the following assumptions (Herakovich, 1998; Tuttle, 2004; Daniel and Ishai 2006):

1. the thickness of the laminate is much smaller than its other dimensions;
2. the laminate and the composing laminae are in a state of plane stresses;
3. the composing laminae are perfectly bonded together;
4. the normal to the middle surface remains straight and normal to that surface after deformation;
5. the normals have constant length; and
6. the displacements are sufficiently small so that quantities of second and higher orders can be neglected.

A typical n-ply laminate in the global x–y–z coordinate system is illustrated in **Figure 3**. Based on the above



(a) Resultant forces and moments in a laminate



(b) Cross-section of an n-ply laminate

Figure 3 Typical n-ply laminate in the global x–y–z coordinate system

assumptions, the strains at any arbitrary point of the laminate are defined by

$$\varepsilon_z = \gamma_{xz} = \gamma_{yz} = 0 \tag{42}$$

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} + z \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \tag{43}$$

where ε_x^0, ε_y^0, and γ_{xy}^0 are the strains at the middle plane, κ_x, κ_y, and κ_{xy} are the curvatures of the laminate.

With Equations (42) and (43), the stresses of each lamina in the x–y plane can be calculated using Equation (22). The resultant forces and moments (**Figure 3**) in the laminate can thus be obtained using the following equation, by summing the contributions of all composing laminae:

$$\begin{Bmatrix} N_x \\ N_y \\ N_{xy} \\ M_x \\ M_y \\ M_{xy} \end{Bmatrix} = \begin{bmatrix} A_{11} & A_{12} & A_{16} & B_{11} & B_{12} & B_{16} \\ A_{12} & A_{22} & A_{26} & B_{12} & B_{22} & B_{26} \\ A_{16} & A_{26} & A_{66} & B_{16} & B_{26} & B_{66} \\ B_{11} & B_{12} & B_{16} & D_{11} & D_{12} & D_{16} \\ B_{12} & B_{22} & B_{26} & D_{12} & D_{22} & D_{26} \\ B_{16} & B_{26} & B_{66} & D_{16} & D_{26} & D_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \\ \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix}$$

$$= \begin{bmatrix} [A] & [B] \\ [B] & [D] \end{bmatrix} \begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \\ \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \tag{44}$$

where N_x, and N_y are the resultant normal forces in the global x- and y-directions per unit width respectively; N_{xy} is the resultant shear force in the global x–y plane per unit width; M_x and M_y are the resultant moments in the global x–z and y–z planes per unit width respectively; M_{xy} is the resultant twisting moment per unit width; $[A]$ is the extensional stiffness matrix, relating in-plane forces to in-plane strains; $[B]$ is the coupling stiffness matrix, relating in-plane forces to curvatures and moments to in-plane strains; $[D]$ is the flexural stiffness matrix, relating moments to curvatures. $[A]$, $[B]$ and $[D]$ are given by

$$A_{ij} = \sum_{k=1}^{n} (\overline{Q_{ij}})_k t_k \tag{45}$$

$$B_{ij} = \frac{1}{2} \sum_{i=1}^{n} (\overline{Q_{ij}})_k (z_k^2 - z_{k-1}^2) \tag{46}$$

$$D_{ij} = \frac{1}{3} \sum_{i=1}^{n} (\overline{Q_{ij}})_k (z_k^3 - z_{k-1}^3) \tag{47}$$

where z_k is the z-coordinate of the upper surface of ply k; t_k is the thickness of ply k; n is the number of composing laminae in the laminate.

A laminate is referred to as a symmetric laminate when for each ply of lamina on one side of the middle plane there is an identical ply stacked at the same orientation and at an equal distance from the middle plane on the other side. A laminate is called a cross-ply laminate when it only consists of plies of laminae with fibre orientations of 0° and 90°; and it is called a balanced laminate when it consists of pairs of identical plies of laminae stacked at $+\theta$ and $-\theta$ orientations respectively.

For a symmetric laminate, the coupling stiffness matrix B is zero; for a cross-ply or balanced laminate, $A_{16} = A_{26} = 0$. Therefore, for a symmetric balanced or symmetric cross-ply laminate, the in-plane force–strain relationship reduces to the following form which is the same as that of an orthotropic material:

$$\begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} = \begin{bmatrix} A_{11} & A_{12} & 0 \\ A_{12} & A_{22} & 0 \\ 0 & 0 & A_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} \tag{48}$$

Such a laminate can thus be treated as an equivalent orthotropic material with effective elastic engineering constants (i.e. elastic modulus, Poisson's ratio and shear modulus). That is, the following equation can be used to represent the force–strain relationship of such a laminate:

$$\begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} = \begin{bmatrix} \dfrac{E_{x,\text{eff}}}{1 - \nu_{xy,\text{eff}}\nu_{yx,\text{eff}}} & \dfrac{\nu_{yx,\text{eff}}E_{x,\text{eff}}}{1 - \nu_{xy,\text{eff}}\nu_{yx,\text{eff}}} & 0 \\ \dfrac{\nu_{xy,\text{eff}}E_{y,\text{eff}}}{1 - \nu_{xy,\text{eff}}\nu_{yx,\text{eff}}} & \dfrac{E_{y,\text{eff}}}{1 - \nu_{xy,\text{eff}}\nu_{yx,\text{eff}}} & 0 \\ 0 & 0 & G_{xy,\text{eff}} \end{bmatrix}$$

$$\times \begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} t \tag{49}$$

where $E_{x,\text{eff}}$ and $E_{y,\text{eff}}$ are the effective elastic moduli; $\nu_{xy,\text{eff}}$ and $\nu_{yx,\text{eff}}$ are the effective Poisson's ratios; $G_{xy,\text{eff}}$ is the effective in-plane shear modulus; and t is the thickness of the laminate.

For an arbitrary laminate, Equation (44) can be inverted to produce

$$\begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \\ \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} = \begin{bmatrix} A_{11} & A_{12} & A_{16} & B_{11} & B_{12} & B_{16} \\ A_{12} & A_{22} & A_{26} & B_{12} & B_{22} & B_{26} \\ A_{16} & A_{26} & A_{66} & B_{16} & B_{26} & B_{66} \\ B_{11} & B_{12} & B_{16} & D_{11} & D_{12} & D_{16} \\ B_{12} & B_{22} & B_{26} & D_{12} & D_{22} & D_{26} \\ B_{16} & B_{26} & B_{66} & D_{16} & D_{26} & D_{66} \end{bmatrix}^{-1} \begin{Bmatrix} N_x \\ N_y \\ N_{xy} \\ M_x \\ M_y \\ M_{xy} \end{Bmatrix}$$

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$$= \begin{bmatrix} a_{11} & a_{12} & a_{16} & b_{11} & b_{12} & b_{16} \\ a_{12} & a_{22} & a_{26} & b_{12} & b_{22} & b_{26} \\ a_{16} & a_{26} & a_{66} & b_{16} & b_{26} & b_{66} \\ b_{11} & b_{12} & b_{16} & d_{11} & d_{12} & d_{16} \\ b_{12} & b_{22} & b_{26} & d_{12} & d_{22} & d_{26} \\ b_{16} & b_{26} & b_{66} & d_{16} & d_{26} & d_{66} \end{bmatrix} \begin{Bmatrix} N_x \\ N_y \\ N_{xy} \\ M_x \\ M_y \\ M_{xy} \end{Bmatrix} \quad (50)$$

If only the in-plane forces are present (i.e. all moments are equal to zero), Equation (50) reduces to

$$\begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} = \begin{bmatrix} a_{11} & a_{12} & a_{16} \\ a_{12} & a_{22} & a_{26} \\ a_{16} & a_{26} & a_{66} \end{bmatrix} \begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} \quad (51)$$

and in terms of the effective elastic constants, Equation (51) can be rewritten as:

$$\begin{Bmatrix} \varepsilon_x^0 \\ \varepsilon_y^0 \\ \gamma_{xy}^0 \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_{x,\text{eff}}} & \frac{-\nu_{yx,\text{eff}}}{E_{y,\text{eff}}} & \frac{\eta_{sx,\text{eff}}}{G_{xy,\text{eff}}} \\ \frac{-\nu_{xy,\text{eff}}}{E_{x,\text{eff}}} & \frac{1}{E_{y,\text{eff}}} & \frac{\eta_{sy,\text{eff}}}{G_{xy,\text{eff}}} \\ \frac{\eta_{xs,\text{eff}}}{E_{x,\text{eff}}} & \frac{\eta_{ys,\text{eff}}}{E_{y,\text{eff}}} & \frac{1}{G_{xy,\text{eff}}} \end{bmatrix} \times \begin{Bmatrix} N_x \\ N_y \\ N_{xy} \end{Bmatrix} \frac{1}{t} \quad (52)$$

where $\eta_{xs,\text{eff}}$, $\eta_{sx,\text{eff}}$, $\eta_{ys,\text{eff}}$ and $\eta_{sy,\text{eff}}$ are the effective shear coupling coefficients.

Additional information

The two-step mechanical analysis presented in this chapter is a widely accepted method for predicting the mechanical properties of FRP composites. In this method, the FRP composite is assumed to be linear elastic. While this assumption is very close to the actual behaviour of most unidirectional FRP composites under uniaxial tension in the fibre direction, non-linear behaviour can become significant under some conditions. For example, extensive research (e.g. Hahn and Tsai, 1973; Lifshitz, 1988; Haj-Ali and Kilic, 2002; Yuan *et al*., 2002) has shown that deviations from linearity occur when an FRP composite is subjected to in-plane transverse loading or in-plane shear, with the latter being more significant. It is also assumed in this method that the elastic constants of FRP composites in tension and in compression are the same. The compressive modulus of elasticity of FRP composites, however, is usually somewhat smaller than the tensile modulus of elasticity (ACI 440.2R, 2008). Therefore, standard material tests (e.g. ASTM D3518/D3518M-94, 1994; ASTM D3410/D3410M-95, 1995; ASTM D2290-00, 2000; ASTM D3039/D3039M-00, 2000; ASTM D695-02a, 2002) should be considered for a more accurate evaluation of the mechanical properties of FRP composites.

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Chapter 55

Failure criteria for FRP composites

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Two common failure modes exist for FRP composites: intralaminar failure and interlaminar failure. The former occurs within a lamina and the latter occurs at interfaces between laminae. Both failure modes are discussed in this chapter. A computational procedure for the prediction of intralaminar failure is presented. The causes of interlaminar stresses which lead to interlaminar failure in a laminate are also explained.

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Introduction

The failure mechanism of fibre-reinforced polymer (FRP) composites is complex and depends on many factors such as the stress state induced by the applied loading, the properties of the constituents (i.e. the fibres and the matrix) and the structure of the composites (i.e. the directions and stacking sequence of the fibres) (Herakovich, 1998; Jones, 1999; Daniel and Ishai, 2006). At the microscopic level, the typical failure modes of an FRP composite include fibre failure (rupture, buckling and splitting), matrix failure, fibre–matrix interface failure and a combination of two or more of these failure modes. Although various micromechanics approaches are available, accurate prediction of the strength or failure of an FRP composite using such approaches is often difficult (Daniel and Ishai, 2006). Consequently, micromechanics approaches have not been widely used for strength predictions despite their popularity in the prediction of mechanical properties of FRP composites. This chapter deals with the failure of FRP composites from a macromechnical point of view. Readers interested in micromechanics approaches should consult Herakovich (1998), Jones (1999) and Daniel and Ishai (2006) for more information. As in Chapter 54, this chapter is concerned only with laminated FRP composites (or FRP laminates) composed of two or more unidirectional laminae.

At the macroscopic level, failure of FRP composites can generally be categorised into two main modes: intralaminar failure (failure inside individual laminae) and interlaminar failure (failure between adjacent laminae such as separation and sliding). The former is caused by stresses and strains in each composing lamina of the laminate, while the latter is caused by interlaminar stresses and strains. The section on Failure criteria for a lamina covers the failure criteria for a single lamina, while the section on Failure analysis of a laminate deals with the failure analysis of a laminate using the classical lamination theory, considering only intralaminar failure. Interlaminar failure is examined in the section on Interlaminar failure of a laminate. Limitations of the theories presented in the chapter are briefly discussed in the section on Additional information.

Failure criteria for a lamina

Due to its orthotropic nature, the strength of an FRP lamina varies with direction. Its tensile strength and compressive strength are also typically different (ACI 440.2R, 2008). Similar to those for an isotropic material, the failure criteria for an orthotropic lamina relate its strength under an arbitrary stress combination to some basic strength parameters. However, the principal stresses and strains are usually not the critical factors in the failure of a lamina due to the dependence of its strength on direction (Jones, 1999). For a lamina subjected to in-plane loading, existing failure criteria are commonly based on the following basic strength parameters which can be found from standard material tests:

- the tensile strength in the fibre direction (longitudinal tensile strength) F_{1t};
- the compressive strength in the fibre direction (longitudinal compressive strength) F_{1c};
- the tensile strength in the direction transverse to the fibres (transverse tensile strength) F_{2t};
- the compressive strength in the direction transverse to the fibres (transverse compressive strength) F_{2c}; and
- the in-plane shear strength F_{12}.

The use of these failure criteria generally requires the transformation of stresses (or strains) in arbitrary directions into the principal material directions. Four most popular failure criteria, i.e. the maximum stress criterion, the maximum strain criterion, the Tsai–Hill criterion, and the Tsai–Wu criterion, are given below. As the composing laminae of an FRP laminate are mainly subjected to in-plane loading in most civil engineering applications, only the simplified form of these four criteria for such loading conditions is presented.

Maximum stress criterion

According to the maximum stress failure criterion, failure occurs when the stress in one of the principal material directions exceeds the corresponding strength (Daniel and Ishai, 2006). This criterion can be described by the following equations:

$$\sigma_1 = \begin{cases} F_{1t} & \text{for } \sigma_1 > 0 \\ -F_{1c} & \text{for } \sigma_1 < 0 \end{cases} \quad (1)$$

$$\sigma_2 = \begin{cases} F_{2t} & \text{for } \sigma_2 > 0 \\ -F_{2c} & \text{for } \sigma_2 < 0 \end{cases} \quad (2)$$

$$|\tau_{12}| = F_{12} \quad (3)$$

where σ_1 is the normal stress in the longitudinal direction; σ_2 is the normal stress in the transverse direction; and τ_{12} is the in-plane shear stress in the principal material coordinate system.

Maximum strain criterion

According to the maximum strain failure criterion, failure occurs when the strain in one of the principal material directions exceeds the corresponding ultimate strain (Daniel and Ishai, 2006). This criterion can be described by the following equations:

$$\varepsilon_1 = \begin{cases} \varepsilon_{1t}^u & \text{for } \varepsilon_1 > 0 \\ -\varepsilon_{1c}^u & \text{for } \varepsilon_1 < 0 \end{cases} \quad (4)$$

$$\varepsilon_2 = \begin{cases} \varepsilon_{2t}^u & \text{for } \varepsilon_2 > 0 \\ -\varepsilon_{2c}^u & \text{for } \varepsilon_2 < 0 \end{cases} \quad (5)$$

$$|\gamma_{12}| = \gamma_{12}^u \quad (6)$$

where ε_1 is the normal strain in the longitudinal direction; ε_2 is the normal strain in the transverse direction; ε_{1t}^u and ε_{1c}^u are the ultimate tensile strain and ultimate compressive strain in the longitudinal direction respectively; ε_{2t}^u and ε_{2c}^u are the ultimate tensile strain and ultimate compressive strain in the transverse direction respectively; γ_{12} is the in-plane shear strain; and γ_{12}^u is the ultimate in-plane shear strain.

For a linear elastic lamina, Equations (3) to (5) can be rewritten as:

$$\sigma_1 - \nu_{12}\sigma_2 = \begin{cases} F_{1t} & \text{for } \varepsilon_1 > 0 \\ -F_{1c} & \text{for } \varepsilon_1 < 0 \end{cases} \quad (7)$$

$$\sigma_2 - \nu_{21}\sigma_1 = \begin{cases} F_{2t} & \text{for } \varepsilon_2 > 0 \\ -F_{2c} & \text{for } \varepsilon_2 < 0 \end{cases} \quad (8)$$

$$|\tau_{12}| = F_{12} \quad (9)$$

where ν_{12} and ν_{21} are the longitudinal (or major) and the transverse (or minor) Poisson's ratios respectively.

Tsai–Hill criterion

The Tsai–Hill failure criterion (Tsai, 1968; Azzi and Tsai 1965) takes the following form:

$$\frac{\sigma_1^2}{F_1^2} - \frac{\sigma_1\sigma_2}{F_1^2} + \frac{\sigma_2^2}{F_2^2} + \frac{\tau_{12}^2}{F_{12}^2} = 1 \quad (10)$$

where F_1 is the strength of the lamina in the longitudinal direction which can be taken to be equal to F_{1t} when σ_1 is positive and to be equal to F_{1c} when σ_1 is negative; F_2 is the strength in the transverse direction which can be taken to be equal to F_{2t} when σ_2 is positive and to be equal to F_{2c} when σ_2 is negative.

Tsai–Wu criterion

The Tsai–Wu failure criterion (Tsai and Wu, 1971) takes the following form:

$$\left(\frac{1}{F_{1t}} - \frac{1}{F_{1c}}\right)\sigma_1 + \left(\frac{1}{F_{2t}} - \frac{1}{F_{2c}}\right)\sigma_2 + \frac{1}{F_{1t}F_{1c}}\sigma_1^2 + 2f_{12}\sigma_1\sigma_2 + \frac{1}{F_{2t}F_{2c}}\sigma_2^2 + \frac{1}{F_{12}^2}\tau_{12}^2 = 1 \quad (11)$$

where f_{12} is a parameter that can be determined from biaxial tests. As biaxial tests are often difficult to conduct, some researchers have suggested that f_{12} may be taken as zero (Narayanaswami and Adelman, 1977) or approximated by the following equation (Daniel and Ishai, 2006):

$$f_{12} = -\frac{1}{2}\sqrt{\frac{1}{F_{1t}F_{1c}F_{2t}F_{2c}}} \quad (12)$$

The maximum stress criterion and the maximum strain criterion are conceptually simple, but each consists of several subcriteria and is thus computationally inconvenient. The interaction among the stress components is not considered at all in the maximum stress criterion, and is only reflected through Poisson's effect in the maximum strain criterion. On the contrary, both the Tsai–Hill criterion and the Tsai–Wu criterion account for stress interaction by providing a single equation to define failure. As the Tsai–Hill criterion (Equation (10)) requires a separate consideration of each of the four quadrants of the stress space, the Tsai–Wu criterion is computationally the simplest.

Figure 1 shows a comparison of the four criteria for a simple case of $\tau_{12} = 0$ for a typical glass FRP lamina. Note that Equation (12) was used to determine the value of f_{12} in plotting **Figure 1**. It is seen that the predictions of the four failure criteria can be significantly different for some loading conditions as indicated by Soden *et al.* (1998). Experimental studies have shown that the maximum stress criterion and the maximum strain criterion lead to significant errors in many cases, especially when both the transverse normal stress σ_2 and the in-plane shear stress τ_{12} are significant. The Tsai–Wu criterion has been shown

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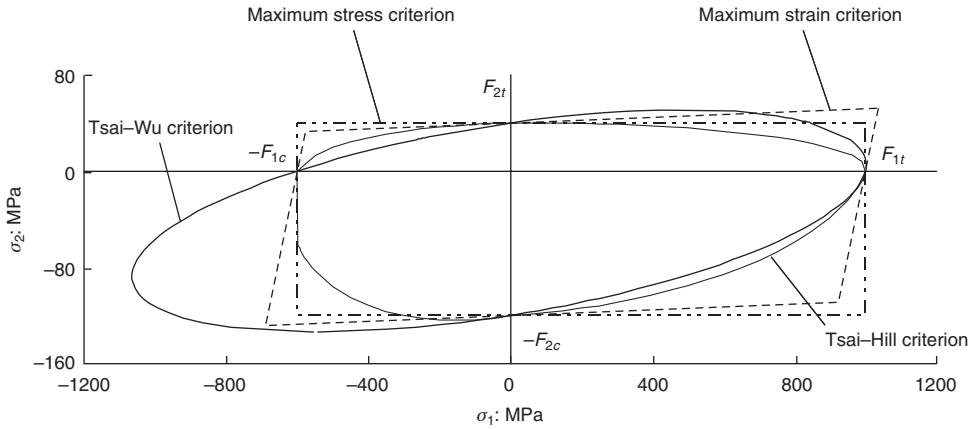


Figure 1 Comparison of four different failure criteria

to have the best performance for general loading cases (Hinton *et al.*, 2002, 2004).

Failure analysis of a laminate

This section presents a method for the failure analysis of a laminate through which the failure process and load-carrying capacity of the laminate can be predicted. When only failure in individual laminae is considered, a failure analysis requires the following tools:

1 an approach to evaluate the stresses/strains in each composing lamina of a laminate under a given loading condition;
2 a lamina failure criterion; and
3 a definition of laminate failure.

Assuming that the internal forces in a laminated section have been determined from a classical structural analysis, the classical lamination theory (CLT) presented in Chapter 54 can then be used to predict the stress or strain state in the laminate. One of the four failure criteria presented in the previous section can be chosen to define the failure of a lamina.

For the failure of laminates, various definitions have been proposed (Tuttle, 2004; Altenbach *et al.*, 2004; Daniel and Ishai, 2006). Among them, the two most popular definitions are the first-ply failure criterion and the ultimate laminate failure criterion. In the former definition, a laminate is deemed to have reached failure once any ply fails. In the latter definition, a laminate is commonly deemed to have reached failure when the maximum load-carrying capacity is attained. These two definitions of laminate failure are presented below.

First-ply failure

With the first-ply failure criterion, failure of a laminate can be predicted using the following procedure (Tuttle, 2004; Altenbach *et al.*, 2004; Daniel and Ishai, 2006):

1 determine the mechanical properties of each lamina either from experiments or using Equations (1a), (1b) and (4–7) from Chapter 54;
2 calculate the stiffness matrix $[Q]$ for each lamina in their respective principal material coordinate system, using Equations (10)–(13) from Chapter 54;
3 calculate the stiffness matrix $\overline{[Q]}$ of each lamina in the reference coordinate system of the laminate, using Equations (23)–(28) from Chapter 54;
4 calculate the stiffness matrix $\begin{bmatrix} [A] & [B] \\ [B] & [D] \end{bmatrix}$ of the laminate, using Equations (45)–(47) from Chapter 54;
5 calculate the stress state of each lamina for a specific load level, using Equations (44), (43) and (22) from Chapter 54;
6 check the stress state of each lamina against a selected failure criterion (e.g. Tsai–Wu failure criterion) to see whether any ply has reached failure;
7 the laminate is deemed to have reached failure once any lamina satisfies the failure criterion; otherwise increase the load level and repeat Steps (5)–(7).

The failure load of a laminate evaluated using the above procedure is generally conservative and sometimes overly conservative, especially for multidirectional laminates (Tuttle, 2004; Altenbach *et al.*, 2004; Daniel and Ishai, 2006). This is because the forces originally resisted by the failed lamina can very often be taken over by the other laminae through internal force redistribution and the load carried by the laminate can continue to increase until the ultimate limit state is reached. In addition, first-ply failure is often initiated by cracking in the matrix instead of fibre rupture. The 'failed' lamina in such a case still retains a significant stiffness because of the constraints offered by adjacent plies.

Ultimate laminate failure

The prediction of ultimate laminate failure requires the analysis process described above to continue until the maximum load is reached. The only difficult part in this process is the determination of the contribution of the failed laminae. It is obvious that the elastic properties of a failed lamina are reduced, but controversy still exists on how such reductions should be evaluated. Nevertheless, it is generally accepted that the fibre-dominant properties (e.g. E_1) and the matrix-dominant properties (e.g. E_2 and G_{12}) should experience different reductions (Tuttle, 2004; Daniel and Ishai, 2006), depending on the microscopic failure mode of the lamina (e.g. matrix cracking and fibre rupture). A simple approach is to totally ignore the affected properties while keeping the other properties unchanged. Based on this simple treatment, ultimate laminate failure can be predicted using the following procedure (Tuttle, 2004; Altenbach *et al.*, 2004; Daniel and Ishai, 2006):

1–6 as described above for the prediction of first-ply failure;
7 increase the load and repeat Steps 5–7 if no lamina satisfies the chosen failure criterion; otherwise go to Step 8;
8 calculate the reduced stiffness matrix of the failed lamina;
9 recalculate the stiffness matrix of the laminate;
10 recalculate the stress state of each lamina;
11 check the stress state of each undamaged lamina against the selected failure criterion to determine whether any additional ply has reached failure;
12 repeat Steps 8–11 if any additional ply has reached failure; otherwise go to Step 13;
13 increase the load level and repeat Steps 5–7 until the next ply fails;
14 continue Steps 8–13 until the maximum load is reached.

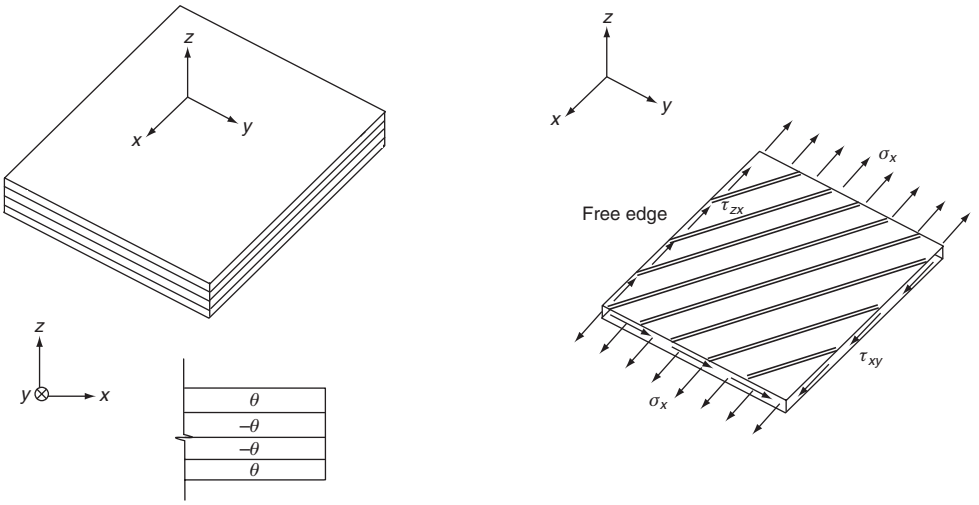
Although the above widely used procedure allows ultimate laminate failure to be predicted, it may lead to considerable errors as the assumptions made in the CLT become increasingly invalid as more laminae fail (Tuttle, 2004). In addition, errors may arise from inaccuracy in evaluating the contribution of the failed laminae.

Interlaminar failure of a laminate

The interlaminar failure of a laminate is caused by the interlaminar stresses, including interlaminar normal (peeling) stresses initiating the separation of the bonded laminae and interlaminar shear stresses initiating the sliding between adjacent laminae. The interlaminar stresses cannot be predicted by the CLT which assumes a plane stress state. These stresses typically occur in the vicinity of free edges (i.e. edges without external forces) (Powell, 1994; Herakovich, 1998; Jones, 1999; Tuttle, 2004) and are thus also referred to as 'free-edge stresses'. The free-edge stresses are generally caused by a mismatch in the properties of adjacent laminae. Two simple cases (Powell, 1994; Jones, 1999; Daniel and Ishai, 2006) are explained as follows.

Mismatch in the shear coupling coefficient

Figure 2 shows a symmetric balanced laminate composed of four identical laminae stacked at $+\theta$ and $-\theta$ directions. According to Equations (29)–(41) from Chapter 54, the



(a) Stacking sequence of the laminate

(b) Generation of free-edge stresses on the upper ply θ

Figure 2 Free-edge stresses in a symmetric balanced laminate

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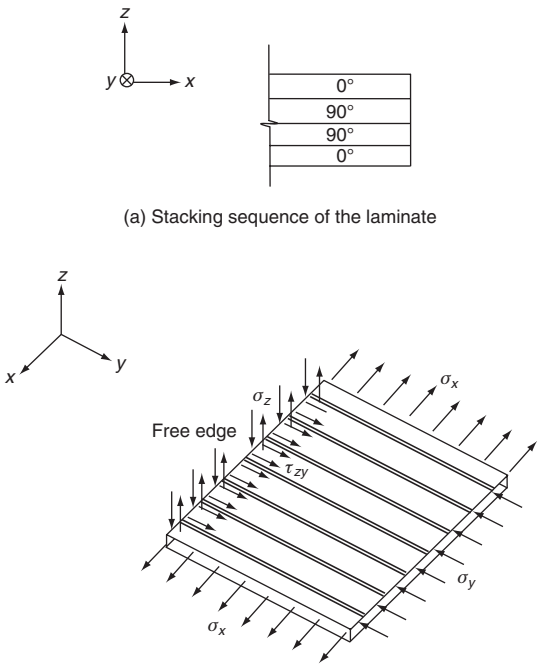
shear coupling coefficients of the $+\theta$ plies and the $-\theta$ plies have the same absolute value but opposite signs in the x–y coordinate system. When subjected to a uniform tensile stress in the x direction, the $+\theta$ plies and the $-\theta$ plies independently tend to induce shear deformations in two opposite directions. In a laminate, this mismatch in the shear coupling coefficient necessitates the development of shear stresses τ_{xy} in order to satisfy deformation compatibility. The existence of τ_{xy} can be predicted by the CLT. However, in reality, these shear stresses cannot exist on the free edges of a laminate. The satisfaction of force equilibrium thus requires the development of the interlaminar shear stress τ_{zx} near the free edges.

Mismatch in Poisson's ratio

Figure 3 shows a symmetric cross-ply laminate composed of four identical laminae stacked at 0° and 90° directions. It is obvious that the Poisson's ratios of the 0° plies and the 90° plies are different. When subjected to a uniform tensile stress in the x direction, the 0° plies and the 90° plies independently tend to deform differently in the y direction. In a laminate, this mismatch in the Poisson's ratio induces transverse stresses σ_y in order to satisfy deformation compatibility. The existence of σ_y can be predicted by the CLT. However, these transverse stresses cannot exist in reality on the free edges. The satisfaction of force equilibrium thus requires the development of the interlaminar shear stress τ_{zy} near the free edges. The shear stress τ_{zy} near the free edges and the σ_y elsewhere together result in a bending moment which has to be balanced by the interlaminar normal stress σ_z near the free edges.

In an arbitrary multidirectional laminate, both types of mismatches exist and a combination of σ_z, τ_{zy} and τ_{zx} can be generated. While the above explains in principle the generation of interlaminar stresses, accurate evaluation of these stresses is rather complex and generally requires the use of numerical methods (Daniel and Ishai, 2006). Many numerical studies have been conducted in this area and they generally confirm that the interlaminar stresses are limited to a small region near the free edges. Further information on interlaminar stresses may be found in Pipes and Pagano (1971), Rybicki (1971) and Herakovich *et al.* (1976).

Interlaminar stresses in FRP laminates also depend significantly on the stacking sequence of the laminae (Jones, 1999; Pagano and Pipes, 1971). Their magnitude can thus be controlled through a careful design of the structure of the laminate. For example, laminae of similar fibre orientations (either $+\theta$ or $-\theta$) should be dispersed throughout the thickness of the laminate instead of being put together to reduce the magnitude of interlaminar stresses (Jones, 1999). A laminate of the 15°/45°/−45°/−15° stacking sequence is much better than a laminate of the 15°/−15°/45°/−45° stacking sequence (Pagano and Pipes, 1971). In practice, interlaminar failure can also be controlled by providing fastening or stitching near the free edges.



(a) Stacking sequence of the laminate

(b) Generation of free-edge stresses on the upper 90° ply

Figure 3 Free-edge stresses in a symmetric cross-ply laminate

Additional information

As pointed out earlier, the method of failure analysis for FRP composites presented in the section on Failure analysis of a laminate may lead to considerable errors, especially for the prediction of ultimate laminate failure. This is mainly because the nonlinear behavior of FRP composites after the first-ply failure is not appropriately addressed in the method. This nonlinearity stems mainly from either the ductility of the constituent materials (especially the matrix) or the progressive deterioration of their mechanical properties due to the evolution of damage, and has a significant effect on the failure process of the composite (Herakovich, 1998; Tuttle, 2004). This issue needs to be appropriately considered for a more accurate prediction of the failure process of FRP composites. Many studies have been conducted in this direction, making use of damage mechanics approaches. Further information on this topic may be found in Wu (1968), Ladeveze (1992) and Orifici *et al.* (2008).

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Chapter 56

Codes and standards

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The availability of an infrastructure based on design codes, test method standards and material specifications is crucial for the growth of composite material products and applications, particularly in industry sectors with a high reliance on codes, such as construction. Examples are given of the underpinning materials test methods/specifications and codes that are being developed relevant to construction applications.

Assumed as already covered: polymer matrix composites (PMC), fibre-reinforced plastics (FRP), carbon-fibre-reinforced plastics (CFRP), glass-reinforced plastics (GRP), glass-fibre-reinforced plastics (GFRP).

Introduction

The availability of an infrastructure based on design codes, test method standards and material specifications is crucial for the growth of the market for composite material products, particularly in industry sectors with a high reliance on codes, such as construction. This infrastructure is particularly important for the growth of the many small to medium enterprise (SME) companies encompassing designers, processors, fabricators and users of these materials that cannot afford the cost of individual qualifications for each project. This chapter concentrates on examples of the infrastructural codes being developed in support of formal design procedures for composite materials products, particularly to meet regulatory requirements.

Developments in the test methods and the product standards are often concurrent. There are often delays in implementing the new test methods in the product standards unless there is good liaison between the responsible committees. Equally, it is of interest in encouraging the greater use of composite materials by engineers that there is consistency in the design approach used (the choice of performance- or prescriptive-based approaches, best practice adopted in detail design for bolted joints, fatigue lifing procedures, etc.).

Polymer composites – material standards

Constituent material specifications and test methods

There is now a fairly extensive standards infrastructure for textile-size fibres (i.e. can be woven into fabrics), initially covering glass fibres and similar standards for carbon-fibres, as shown in **Table 1**. More recent standardisation has been aimed at producing a generic version of each of these test methods. For example, linear density is essentially independent of fibre type, and a generic standard can cover current fibres and, importantly, new textile-size fibres, without the need to draft, ballot and publish a new specific standard for each new fibre developed.

Resin matrices, both thermoplastic and thermoset, are mainly covered by standards for unreinforced plastic and are not listed here. In general, these methods and specifications are comprehensively represented in the International Standards Organization (ISO) series.

Current work in Comité Européen de Normalisation (CEN) TC 249/WG15 is aimed at developing a specification that will draw together all the specification requirements for the raw materials – fibre, resin, veil and core materials. It is likely to effect a non-change requirement, so that users are advised of any changes. In addition, work is progressing on test methods specifically targeted at fire performance.

Compound specifications and test methods

This area covers the pre-impregnate (pre-preg) or moulding compound when the matrix has been added to the fibre format (see **Table 2**). The matrix can be principally thermoplastic or themoset. For laminated systems, the specification relates to a layer of the material and covers, for example, the amount of resin and volatiles in the pre-preg. The final components in these cases are built from combining multiples of these layers, including combinations of formats. The manufacturing processes used can vary from hand lay-up to high-pressure/high-temperature autoclaves, with corresponding increases in the material quality and reduction in the variability.

Other standards for glass-mat thermoplastics (GMTs) and SMCs/BMCs/DMCs (sheet/bulk/dough moulding compounds) specification (i.e. EN 13677 and EN 14598) relate to moulding compounds that are pressed to obtain the final shape from a dough or a minimum number of layers. These standards establish bounds for the mean properties as agreed between supplier and moulder/user.

Standard No.	Title
ISO 1887	Glass fibre – determination of combustible matter content (size)
ISO 1888	Textile glass – staple fibres or filaments – determination of average diameter
ISO 1889	Reinforcement yarns – determination of density
ISO 1890	Reinforcement yarns – determination of twist
ISO 2078	Textile glass – yarns – designation
ISO 3341	Textile glass – yarns – determination of breaking force and breaking elongation
ISO 9163	Textile glass rovings – preparation of test specimens and determination of tensile strength of impregnated rovings
ISO 9291	Textile glass reinforced plastics – rovings – preparation of unidirectional plats by winding (being replaced by ISO 1268)
ISO 10119	Carbon fibre – determination of density
ISO 10371	Reinforcement materials – braided tapes for composite materials reinforcement – basis for a specification
ISO 10548	Carbon fibre – determination of the size content
ISO 11566	Carbon fibre – determination of tensile properties of single-filament specimens
ISO 11567	Carbon fibre – determination of filament diameter and cross-sectional area
ISO 10618	Carbon fibre – determination of tensile properties of resin-impregnated yarn
ISO 13002	Carbon fibre – designation system for filament yarns
ISO 15039	Textile-glass rovings – determination of solubility of size
ISO 15100	Plastics – reinforcement fibres – chopped strands – determination of bulk density
EN 13003	Para-aramid fibre filament yarns
EN 13417	Specification for woven fabrics – Parts 1–3
EN 13473	Specification for multi-axial multi-ply fabrics (non-crimped fabrics – NCFs) – Parts 1–3
EN 14020	Specification for textile glass rovings – Parts 1–3
EN 14118	Specification for textile glass mats – Parts 1–3
BS 4045	Specification for epoxide resin pre-impregnated glass-fibre fabrics

Table 1 Constituent fibre specifications and test method standards

This approach is to be expected as many 'formulation' variations exist for these materials. The standard defines the properties to be controlled and the limits, in percentage terms, while the customer/supplier agree the absolute levels of the properties.

Importantly, standards are available covering the mouldability/flow and cure properties of these materials prior to final moulding. These properties need to be measured, so that the production quality can be monitored, but are also useful to assess the shelf-life of the materials.

Coupon level test methods

Test panel preparation and quality assessment

The revision of ISO 1268 as a multi-part standard covers most established process routes, as noted in **Table 3**. The aim of this standard is to ensure that test panels are made

Standard No.	Title
ISO 9782	Plastics – reinforced moulding compounds and prepregs – determination of apparent volatile matter content
ISO 10352	Fibre-reinforced plastics – moulding compounds and prepregs – determination of mass per unit area
ISO 11667	Fibre-reinforced plastics – moulding compounds and prepregs – determination of resin, reinforced fibre and mineral filler content – dissolution methods
ISO 12114	Fibre-reinforced plastics – thermosetting moulding compounds and prepregs – determination of cure characteristics
EN ISO 12115	Fibre-reinforced plastics – thermosetting moulding compounds and prepregs – determination of flowability, maturation and shelf life
ISO 15034	Composites – prepregs – determination of resin flow
EN 1342	SMC/BMC – determination of anisotropy
EN 2833	Aerospace: reinforced plastics – glass-fibre pre-impregnates
EN 12575	Plastics – thermosetting moulding compounds – determination of the degree of fibre wet out in SMC
EN 13677	Reinforced-plastics composites – specifications for thermoplastic moulding compounds (GMT)
EN 14447	Fibre-reinforced plastics – glass mat reinforced thermoplastics (GMT) – determination of flowability and solidification
EN 14598	Reinforced plastics composites – specifications for thermoset moulding compounds (SMC, BMC, DMC) – Parts 1–3

Table 2 Compound specifications and test methods

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Part No.	Title
1	General principles
2	Contact and spray-up moulding
3	Wet compression moulding
4	Moulding of preimpregnates
5	Filament moulding
6	Pultrusion moulding
7	Resin transfer moulding
8	Moulding of SMC/BMC
9	Moulding of GMT/STC
10	Injection moulding of BMC/DMC

Table 3 Parts of (EN) ISO 1268 for test plate manufacture

consistently and in a representative manner for the wide range of available manufacturing routes. As new processes outside the existing scopes are established, further parts will be added to the standard. Supporting standards at this level are those related to quality aspects, such as for measuring fibre and void content. For glass-fibre-based systems, ISO 1172 and ISO 7822, based on resin burn-off, are used. For carbon-fibre-based systems, ISO 14127 provides several methods including the principal method of chemical dissolution.

Measurement of T_g, the glass transition temperature, is frequently used for ensuring that cure has been successfully achieved. ISO 6721-11 covers dynamic mechanical analysis (DMA) methods, including calibration of DMA equipment. An alternative method frequently used by resin suppliers is differential scanning calorimetry (DSC) according to ISO 11357. These two techniques are routinely used for high-performance pre-impregnated composites (prepregs), and are often interchanged or used in comparison, although the relationship between these two very different tests is not clear.

However, not all users can afford to have both techniques in-house. These methods have been compared in NPL Measurement Good Practice Guide No. 62 (Mulligan *et al.*, 2000) on thermal analysis methods. Other methods that are more suitable for process control, such as ultrasonics and dielectrics, are not covered by current standards.

Laminate test methods

Table 4 lists the range of test methods available for characterising the cured and/or finally pressed laminate. These are

Standard No.	Title
EN ISO 75-3	Plastics – determination of temperature of deflection under load – high-strength thermosetting laminates and long-fibre-reinforced plastics
EN ISO 527-1	Plastics – determination of tensile properties – Part 1: general principles
EN ISO 527-4	Determination of tensile properties – Part 4: test conditions for isotropic and orthotropic fibre-reinforced plastic composites
EN ISO 527-5	Plastics – determination of tensile properties – Part 5: test conditions for unidirectional fibre-reinforced plastic composites
EN ISO 1172	Textile glass-reinforced plastics – determination of loss on ignition
ISO 1268	Fibre-reinforced plastics – method of producing test plates
EN ISO 2818	Plastics – preparation of test specimens by machining
ISO 3597 (Parts 1–4)	Textile glass reinforced plastics – determination of mechanical properties on rods made of roving-reinforced resin (preparation of rods, flexure, tension and shear strengths)
ISO 4899	Textile glass-reinforced thermosetting plastics – properties and test methods
EN ISO 6721	Plastics – determination of dynamic mechanical properties
EN ISO 10350-2	Plastics – acquisition and presentation of comparable single-point data – Part 2: long-fibre–reinforced plastics
ISO 11357	Plastics – differential scanning calorimetry (multiple parts)
ISO 13003	Fibre-reinforced plastic composites – determination of fatigue properties under cyclic loading
EN ISO 14125	Fibre-reinforced plastics composites – determination of flexural properties
EN ISO 14126	Fibre-reinforced plastic composites – determination of the in-plane compression strength
ISO 14127	Composites – determination of resin, fibre and void content of composites reinforced with carbon fibre
EN ISO 14129	Fibre-reinforced plastic composites – determination of the in-plane shear stress/shear strain, including the in-plane shear modulus and strength by the ±45° tension test method
EN ISO 14130	Fibre-reinforced plastic composites – determination of apparent interlaminar shear strength by short-beam method
ISO 15024	Mode i interlaminar fracture toughness G_{Ic} of unidirectional fibre-reinforced polymer matrix composites
EN ISO 15310	Fibre-reinforced plastic composites – determination of in-plane shear modulus by plate twist
ISO 18352	Test method for compression-after-impact properties of carbon-fibre-reinforced plastics
ISO 22314	Glass-reinforced products – determination of fibre length

Table 4 Composite material test methods

Property	International standard	ASTM methods	CRAG methods
Tension – 'isotropic' (nominally)	EN ISO 527-4	D 3930	300
Tension – unidirectional (anisotropic)	EN ISO 527-5	D 3930	300
Flexure	EN ISO 14125	D 695	200
Compression	EN ISO 14126	D 3410	400
Shear – $\pm 45^{\circ}$ tension	EN ISO 14129	D 3815	101
Shear – interlaminar by short beam flexure	EN ISO 14130	D 2344	100

Table 5 Harmonized mechanical test methods (N.B. not fully equivalent)

mainly mechanical tests or tests for assessing quality in terms of void and fibre content and degree of cure. The latter is particularly important as the material in most cases (excluding pultruded profiles) only becomes the fully performing material as the product itself is formed.

The comprehensive series of mechanical test methods now available followed extensive harmonisation in the 1990s. Consequently, there is a high level of similarity between ISO, EN – general series, ASTM (American Society for Testing and Materials) standards and unofficial codes, such as the CRAG methods. Some of the laminate test methods in different series are listed in **Table 5**.

These test methods have been brought together to provide a Standards Qualification Plan (or SQP) (Gower and Sims, 2003) developed in the UK with wide industrial support. This covered the data to be obtained and the test conditions, including batch sizes and non-ambient temperatures. The precision of several of the critical test methods was evaluated in an experimental round-robin exercise to gain confidence and mutual recognition of each others' data. A second document, entitled the Extended Qualification Plan (EQP), extends the SQP to cover more complex properties, such as due to stress concentrations (e.g. bearing properties, open-hole compression and tension, barely visible impact damage), that are currently ISO work items. It is planned to submit the SQP for international standardisation. Non-mechanical test methods are discussed below.

Composite sub-components

There is only one example of a finished (i.e. cured) 'off-the-shelf' composite material, which is pultruded profiles. They mirror metal extrusions and are fabricated in several thin-walled shapes such as tees, channels and boxes. These are extensively used as the basic building units for access structures (e.g. ladders and walkways, as discussed later in the following section) and other structures (e.g. bridges). For pultrusions, the CEN specification, EN 13706 for pultruded profiles, has increased the application and sales of pultruded sections. The standard defines test methods and minimum levels of properties for, initially, two grades.

The standard is structured as follows:

EN 13706 Pultruded profiles – specifications
- Part 1 – Designation
- Part 2 – Test methods and general requirements
- Part 3 – Detailed specifications.

The designation provides a code that represents the basic nature of the pultruded profile such as fibre and resin type, profile shape and grade. The mandated requirements refer to properties measured using the laminate test methods listed above or in annexes to Part 2 (i.e. bearing strength). Part 2 also covers important practical aspects of tolerances and defects, together with recommended test methods for non-mechanical properties, such as durability. Part 3 defines the minimum requirements to qualify as an E17 or E23 grade profile. The E17/E23 refer to the material modulus of the section determined from a bending test on a full-size section and knowledge of the section moment of inertia. The test span of 50 times section depth is chosen to limit the additional deflection due to shear (cf. deflection due to flexure).

It is possible to add further grades to Part 3. For example, a grade appropriate for use as strengthening 'plates' on bridges and other structures that is based on carbon fibre rather than on glass fibre, as currently in the E17 and E23 grades.

It is of interest to note that the first public composite road bridge (the West Mill Bridge over the River Cole, Oxfordshire) was opened on the B4508 road. Interestingly, the main structural load-bearing units forming the cross-beams are based on plain pultruded GRP box sections, conforming to the E23 grade defined by the EN 13706 standard, stiffened by carbon-fibre cappings. An EU research project funded the development of the more complex-shaped pultrusions known as 'Asset' units that are bonded together to form the road deck. Pultruded profiles can also be fabricated with complex cross-sectional profiles.

A frequent area of concern relates to fire performance. Some of the different test methods used are:

- ISO 5660 (cone calorimeter) and ISO 5658-2 (lateral flame spread) – used in research and development.

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- EN 13823 (SBI test), ISO 5659-2 (smoke opacity plus additional toxicity measurements) – regulatory tests for classification purposes.
- IEC ISO 60695-11-10 (50W flame test) and IEC ISO 60695-11-20 (500W flame test) – bench-scale characterisation and factory production control tests.

For construction products, reference is made to EN 13501 Part 1 – Reaction to Fire and Part 2 – Fire Resistance, which define seven levels of fire performance. One of the test methods used in these standards is EN 13823 (Single Burning Item – SBI), which is not suitable for 'linear' products such as pipes, cables and pultruded profiles. At the time of writing (2009) negotiations are ongoing regarding forming the two panels (i.e. $1\,m \times 1\,m$ and $1\,m \times 0.75\,m$) needed for this test through bolting together box or channel profiles. Agreement is still needed regarding several detailed aspects related to the construction and mounting of the bolted panel.

Product standards

Several bodies are involved in developing product specifications and approvals. Within Europe the main source is CEN in support of European Directives. There are several committees working on product standardisation, which may be for a composites product only (e.g. GRP pressure vessels – CEN/TC 210; piping – ISO TC 138/CEN TC 155) with equivalent standards covering use of other materials in the same application, or an area where a single standard covers all competing materials (e.g. access engineering, i.e. ladders, walkways and handrail – CEN/TC 114). In addition, a standard for offshore GRP piping has been prepared under ISO/TC67 based on the guidance document developed by the UK Offshore Operators' Association.

Product standards, as for access engineering, open to all materials that can meet the technical requirements (e.g. maximum acceptable deflections under prescribed loads) provide opportunities for composites. Performance-based standards are preferred to prescriptive standards that either prevent completely the use of composites or inhibit the design freedom necessary to obtain the cost-effective solution using composite materials. It is important that the development of these performance standards is monitored to ensure that they do not include any requirement that prevents unnecessarily a composite-material-based design solution.

It is important in order to encourage the greater use of composite materials by engineers that there is consistency in the design approach used (e.g. performance- or prescriptive-based approaches, bolted joint design, lifing procedures), unless there are identifiable reasons for any differences. In this section the following composite product standards, related generally to different areas of construction, are reviewed:

- GRP pressure vessels – BS 4994
- GRP water tanks – EN 13280
- Access engineering – EN 14122
- FRP lighting columns – EN 40-7
- GRP rockbolts – BS 7861
- Inspection chambers – BS 7158.

GRP pressure vessels – EN 13121

The British Standard for GRP pressure vessels, BS 4994, was a leading document in 1973 as one of the first 'design' standards for a reinforced plastics/composites product. BS 4994 was basically a prescriptive document that set down minimum requirements representative of current practice at that time. The document was revised in 1987 when several parts were extended, but the basic structure and approach remained unaltered. CEN Technical Committee (TC) 210 has been responsible for developing a European standard for these products, EN 13121, with input from, among others, the existing UK (i.e. BS 4994) and German documents. The work on the new EN standard supports the EC Pressure Equipment Directive (97/23/EC).

The EN standard is divided into four parts:

Part 1 – Raw materials – acceptance and usage conditions
Part 2 – Chemical resistance
Part 3 – Design, calculation and workmanship
Part 4 – Delivery, installation and maintenance.

Part 1 has comprehensive references to ISO test methods and specifications for the constituent materials. Part 2 recognises the major need to quantify the chemical resistance in view of the planned use of many of these pressure vessels. Chemical resistance can be obtained principally through a welded thermoplastic sheet lining or resin-rich glassfibre/resins inner laminates.

The design of GRP pressure vessels is comprehensively covered in Part 3 (>200 pages). The main development from BS 4994 is the inclusion of a design procedure based on test results and performance requirements, rather than the prescriptive requirements in the BS standard and changes to the design strains allowed. Part 4 covers handling, installation and repair issues in more detail than most other documents. These are important aspects that are not always covered in sufficient detail, in order to advise those not used to handling composites of best practice.

Two related standards cover underground storage of petroleum (EN 976 Parts 1–4) and filament wound pressure vessels (EN 13923). The latter, in particular, uses extensive cross-references to EN 13121.

Design approach	Conditions
Advanced design	Allows design from measured data based on batch sizes >15 Lower design factors (cf. basic method) to cover manufacture route, chemical resistance, design temperature/resin HDT ratio, cyclic and long-term loading Minimum design factor = 4 (or 5 in specified cases)
Basic design	Design from properties defined in standard with support from small batch (~5) testing Higher design factors (cf. advanced design) to cover manufacture route, chemical resistance, design temperature/resin HDT ratio, cyclic and long-term loading. Minimum design factor = 6 (data measured in last 12 months accepted) or = 8 (data measured in last 18 months accepted)

Table 6 Comparison of design approaches planned for

The design process is based on the evaluation of both internal loads (e.g. pressure, static head and bending moments) and external factors (e.g. seismic loads, snow and wind loads). The load analysis is checked against the allowable strain levels. The main features of the design procedures are given in **Table 6**. Detailed design guidance is given on:

- conical, dished, flat, ellipsoidal and domed ends;
- knuckle radii, lugs, fixing points, saddle supports;
- seam joints, buckling failures;
- integral branch pipes and bolted flanges;
- panel (square or circular) design, including with stiffeners.

The panel analysis is similar to the procedures available in the NPL CoDA PC Windows software for material property synthesis and preliminary design (Sims and Broughton, 1999). Panel deflection is limited to 1.5 times the panel thickness. Verification tests on prototype design include:

- measurement of local strains when vessel is pressurised,
- determination of fatigue strength,
- determination of maximum failure load (check on safety factor) and failure mode.

The other major design input is the allowable limiting strain (e_{az}), which is equal for each type of resin to 10% of the resin tensile elongation to failure, and for each type of reinforcing material it is determined by the following equation:

$$e_{az} = u_z/(KX_2)$$

where u_z is the ultimate tensile unit strength (UTUS) and X_2 is the unit modulus from the given minimum value or from test data. An allowable design unit loading (u_z) for each type z of reinforcing material is equal to $e_d \times X_z$, where e_d is the design strain. These design strains are limited by the maximum value allowed, which is proposed to be 0.25% (cf. 0.2% in BS 4994), with a higher allowance for short-term overloads (i.e. lasting less than 30 min) of 0.4% if using resins with elongation to failure greater than 2%.

GRP piping – offshore – ISO 14692

ISO 14692, under ISO 67, was formed initially on GRP piping for offshore use and draws heavily from the document produced by the UK Off-shore Oil Operators' Association (UKOOA). This document was well accepted and has been the basis of company documents, where the scope had been extended to cover on-shore and chemical uses. The standard is likely to have a long-term significance for the future acceptance of GRP piping.

The standard has been prepared in four parts that show some agreement with the structure of EN 13121. The parts are:

- Part 1: Applications and materials – mainly covers scope, terminology and material limits.
- Part 2: Qualification and manufacture – mainly covers test and qualification requirements.
- Part 3: System design – covers design procedures.
- Part 4: Fabrication, installation and operation – covers aspects relevant to meeting specified performance requirements.

Pipes with wall thicknesses up to 0.1 × pipe diameter are within the scope of the standard, with a minimum wall thickness of 3 mm for pipe internal diameters less than 100 mm and greater than 0.025 × pipe internal diameter for larger sizes.

Water piping – BS 7159/EN 1115, 1636 and 1796

The GRP piping standard, BS 7159, is a document of similar age and coverage to BS 4994, but places increased emphasis on filament winding as a more frequently used process route. Being used for the same application areas as the pressure vessels, similar approaches are taken to the design piping. This standard, and others such as BS 5480, will be replaced by international standards generated through ISO TC 138 and CEN TC 155. Relevant standards include:

- EN 1115 Plastics piping systems for underground drainage and sewage under pressure. Glass-reinforced thermosetting plastics (GRP) pipes based on unsaturated polyester (UP) resins.

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Standard No.	Title
ISO 7509	Plastics piping systems – glass-reinforced thermosetting plastics (GRP) pipes – determination of time to failure under sustained internal pressure
ISO 7511	Test methods to prove the leak tightness of the wall under short-term internal pressure (see also BS EN 1229)
ISO 7684	Determination of the creep factor under dry conditions (see also BS EN 761)
ISO 7685	Determination of initial specific ring stiffness (see also BS EN 1228)
ISO 8483	Test methods to prove the design of bolted flange joints (see also BS EN 1450)
ISO 8513	Determination of initial longitudinal tensile properties (see also BS EN 1393)
ISO 8521	Determination of the apparent initial circumferential tensile strength (see also BS EN 1394)
ISO 8533	Test methods to prove the design of cemented or wrapped joints
ISO 8639	Test methods for leak tightness and resistance to damage of flexible and reduced-articulation joints (see also BS EN 1119)
ISO 10466	Test method to prove the resistance to initial ring deflection (see also BS EN 1226)
ISO 10468	Determination of the long-term specific creep stiffness under wet conditions and calculation of the wet creep factor (see also BS EN 1225)
ISO 10471	Determination of the long-term ultimate bending strain and the long-term ultimate relative ring deflection under wet conditions
ISO 10928	Methods for regression analysis and their use (see also BS EN 750)
ISO 10952	Determination of the resistance to chemical attack from the inside of a section in a deflected condition (see also BS EN 1120)
ISO 14828	Determination of the long-term specific ring relaxation stiffness under wet conditions and calculation of the wet relaxation factor
EN 1638	Test method for the effect of cyclic internal pressure
EN 1862	Determination of the relative flexural creep following exposure to a chemical environment

Table 7 Test methods for plastics piping systems – glass-reinforced thermosetting plastics (GRP) pipes and fittings

- EN 1636 (six parts) Plastics piping systems for non-pressure drainage and sewage under pressure. Glass-reinforced thermosetting plastics (GRP) pipes based on unsaturated polyester (UP) resins.
- EN 1796 (six parts) Plastics piping systems for water supply with or without pressure. Glass-reinforced thermosetting plastics (GRP) pipes based on unsaturated polyester (UP) resins (see also ISO/DIS 10639).

The ability to develop these product standards relies on available specific product or sub-component test methods as listed in **Table 7** for pipes and pipe sections.

It is important that the work in CEN, ISO and ASTM on GRP piping, including offshore piping, is well coordinated and, at least, aware of the work content in each case to avoid duplication and unnecessarily different material specifications and design procedures.

GRP water tanks – EN 13280 (similar to BS 7491)

CEN standard EN 13280, published in 2000, covers one-piece (500 litre to 100 000 litre) and sectional tanks (500 litre and larger) in support of EU Directive 80/778/EEC. The standard has been developed by a working group within TC 210. A major input into the standard came from BS 7491-3, as this product is particularly found in the UK market. Specific aspects due to the application are the measurement opacity to ISO 3474 and meeting the Drinking Water Directive 80/778/EEC, as these tanks are used for potable water applications.

Access engineering – EN 14122 (four parts)

CEN TC 114 is producing a standard, EN 14122 (four parts), for access engineering (stairways, stepladders and guardrails) under a mandate related to the Safety of Machinery Directive (see EN 292 Safety of Machinery). There are detailed specifications for loads and dimensions of these systems. The standard allows any type of material to be used that meets the stated technical requirements. The materials used must be selected 'to withstand the foreseeable conditions of use' and to meet the safety objectives of the standard, including resisting corrosion due to the surrounding atmosphere (climate, chemical agents and corrosive gases).

Product design is controlled by the mechanical test requirements of the product itself. For example, the deflection of a handrail at the stanchion and at the midpoint of a handrail span L (in metres) due to an applied horizontal load of 300 N $\times$ L must be less than 30 mm. There should be no permanent deflection after a 1 min loading period. Similar tests exist for ladder stiles and rungs. Other aspects cover avoidance of galvanic action, thermal stresses, slippage and bad design features (sharp edges or water accumulation).

This type of performance standard is helpful to composites as it allows a free choice of suitable materials. The pultruded profiles for which a specification standard, EN 13706, has been published are successfully used in these applications, particularly when there is a requirement to resist aggressive environments.

FRP lighting columns – EN 40-7

The standard EN 40-7 is part of a series dealing with different construction materials under CEN TC 50 (e.g. Part 4 – concrete, Part 5 – steel, Part 6 – aluminium) for lighting columns. These specifications relate to Parts 1 to 3 dealing with definitions, general requirements and dimensions, and design and verification. The scope covers columns up to 20 m tall for post-top lanterns and 18 m tall for side-entry lanterns. As well as meeting the horizontal wind loads (EN 40-3), it must also meet the performance in impact under the Essential Requirements No. 4 Safety in Use.

Buckling equations are given as the application is based on thin-walled circular or hexagonal section tubes. These calculations rely on values for the flexural modulus in the longitudinal and transverse direction of the column, the in-plane shear modulus and two principal Poisson's ratios. It is suggested that the last three of these values can be obtained from 'industry standard laminate analysis' of the type embodied in the NPL CoDA composite material synthesiser (Sims and Broughton, 1999). This gives rise to two concerns: one is the lack of a definite reference and the second is the applicability of these analyses, which are normally related only to fully aligned plies of the material.

For the prediction of the torsional buckling strength an additional factor to allow for the effect of cut-outs, for access doors, must be justified by adequate (undefined) testing. An Annex Z is included indicating the clause dealing with essential requirements of the EU Construction Products Directives (89/106). This covers horizontal (wind) loading, impact performance and durability, together with the use of these lighting columns in circulation areas. Commission European (CE) marking for product conformity is also covered in this annex. The verification is according to the design procedures given in Parts 2 and 3 of the standard with modification due to the anisotropy of the composite properties, given by equations in Part 7.

GRP rockbolts – BS 7861

This standard contains requirement and test methods for both steel and GRP rockbolts used as strata reinforcement in coalmines. It covers dimensional, material and performance requirements, but not installation. GRP rockbolts must have a minimum diameter of 21.5 mm, a minimum length of 1.2 m and a non-metallic thread of 150 mm. Nuts can be steel or GRP, with 36 mm across flats. Whereas steel rockbolts have a specific alloy composition description, there is not even a clause for the GRP specification, or for the supply of raw material constituents, although these are available as noted earlier. This is a common problem in the specification of composite products.

There are performance requirements covering antistatic (electrical resistance less than 10^9 ohms), fire resistance (persistence of small flame less than 10 s) and mechanical properties (tensile failure load >300 kN, shear failure load >120 kN and bond strength between resin and rockbolt of 245 kN for a bond length of 450 mm). Specific product tests are given in each case.

Inspection chambers – BS 7158

This BS standard covers both plastic and GRP products, and may be submitted as a CEN standard, with the ultimate aim of allowing GRP manhole covers to be specified in EN 124. This later standard relates mainly to cast iron and steel, although other materials are allowed if they meet the specification, which, in being specific, relates more to the existing metal items rather than performance requirements. The material requirements reference BS standards that have ISO equivalents, for the E glass fibre for the main structure. C glass fibre or thermoplastic veils are used on the surface to give increased durability. Aggregate particles, if used in the design, must be between 0.05 mm and 0.5 mm (e.g. graded silica sands). Fillers for controlling flow should be below 0.05 mm size. Quality control requires that interior surfaces should be smooth and continuous, and that parts are free from protruding fibres, voids, pits, bubbles, cracks, blisters or foreign matter that would prevent the performance requirements being achieved (see also EN 13706).

All the testing requirements (vertical and shear loads, pressure loads, etc.) relate to final products tests and the different grades of products. For the vertical loading, displacements should be less than 6% and be free from cracking. Interestingly, there is a prescribed elevated temperature cycling, 2500 cycles, test for cracking or leakage that includes passage of water at 85°C, but no requirement is given for the resin deflection temperature under load (DTUL measured according to ISO 75, previously known as the heat distortion temperature) except indirectly by way of the reference to BS 3532 for specifying unsaturated polyester resins.

Summary of material aspects

A review of the documents highlighted in this section has been undertaken to study the material aspects used in these products. Some trends are becoming established through cross-references that provide a common route to traceability and qualification. Trends identified are given in **Table 8**.

Design guides for composites in construction

Design code developments

Design codes are still being developed for the use of composites in the construction industry. Although there have been many projects undertaken, they are justified individually rather than through a design code, such as a Eurocode.

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Topic	Summary
Constituent materials	A satisfactory basis exists in ISO series for test and specification purposes, which is being developed further
Laminate test methods	Established EN ISO series is being applied as new or revised documents are produced. The mechanical tests, harmonized with ASTM, can replace CRAG test methods
Materials properties – default	Limited data that compare fairly well between different sources. EUROCOMP (Clarke, 1996) has most (manufacturers') data. Other sources need to be referenced for full databases
Materials properties – measured	Wide variation in requirements but including normally tensile strength and modulus
Temperature capability	Important property controlling service temperature capability, but alternative methods for T_g/DTUL give rise to inconsistencies
Water absorption	ISO 62, or similar procedures, used for both basic data and for application-orientated data (e.g. water tanks)
Chemical resistance	Important property for many applications (+ good specific properties). Need for agreed universal procedures, based on ISO 175/EN 13121
Impact resistance	Material level tests follow ISO 6603, or similar, with expected application-relevant tests of final products
Creep behaviour	In products of relevance, centred around product testing for qualification, with material tests or defaults for design
Fatigue and damage behaviour	Similar to creep situation
Flammability	Initial negative industry response as combustible, now replaced by wider and more enlightened use meeting individual requirements
Static electricity	Mainly of interest in offshore industry, where ISO 14629 has most comprehensive information
Production control	Some consistency developing for wet resin production facilities. Type testing is developing well that demonstrates commitment to high and consistent quality
Cure assessment	Still mainly dependent on Barcol hardness, but expected that increase in automated closed mould production will encourage use of other techniques
Non-destructive testing	Limited use and requirements outside extensive use of C-scan ultrasonics for assessing high-performance composites
Tolerances	In most product areas this issue is being covered, which will demonstrate to purchasers commitment to quality
Defects	Some commonality of approach but agreement needed on definitions and procedures for assessment of defect criticality
Approval of laminators	Practical scheme given in prEN 13121 but national approval schemes to be encouraged
Handling, storage and packaging	Guidance in prEN 13121 should be applied more widely to make installers and users aware of possibilities of handling damage
Repair and maintenance	Agreed procedures need to be developed on a universal basis, together with specific product considerations

Table 8 Summary of material aspects in composite product standards

As of 2009, there is no EN Eurocode for design of composites, either as new buildings, bridges, etc, or in strengthening existing constructions. A small expert group has evaluated and collected information to determine if sufficient knowledge has been accumulated to allow drafting of a Eurocode for composites through EN TC250. It has been proposed that it is timely to initiate a project to develop a suitable Eurocode. One of the most comprehensive precursors was prepared by the EUROCOMP consortia working under the Eureka programme, which produced a major document, consisting of 750 pages (Clarke, 1996).

EUROCOMP – structural design of polymer composites

The EUROCOMP manual was developed in line with the Eurocodes for construction, which are now prominent in Europe (Clarke, 1996). The code specifically deals with glass-fibre-reinforced systems and includes, in particular, the structural use of pultruded profiles, although the principles are to be applied more widely. The EUROCOMP manual is in three parts, with the 'code' itself in Part 1, the handbook in Part 2 providing additional support and explanations, followed by Part 3 containing some research reports. The clauses of the code are divided into 'Principles' stating necessary requirements and 'Applications' that meet the requirements, but other approaches can be used if shown to match the requirements. Part 2 covers much of the ground described in the introduction regarding 'design' textbooks.

The main design rules are based on the Eurocodes for construction, with numerical values taken from Eurocode 1 for the applied 'actions' such as the applied loads, including self-weight, imposed loads and accidental loads (impact from vehicles). These aspects are not reviewed in detail as being common requirements, except where the special nature of composites has some influence.

The structure is designed to reach a serviceability limit state (e.g. excessive deflection, buckling/wrinkling, local damage, environmental damage) before an ultimate limit state is reached resulting in catastrophic collapse of the structure or danger to persons in the vicinity. The design situations identified are:

- normal or persistent conditions of use;
- transient situations (e.g. construction or repair);
- accidental situations.

Actions are described as direct when a load is applied to the structure, or indirect if, for example, due to temperature

effects or settlement. The actions may also be permanent (G) due to self-weight, etc., variable (Q) due to wind loads, etc.; or accidental (A) from blasts or vehicle impact. The Eurocode gives characteristic values (F_k) for these actions. The design factor, F_d, equals F_k times the γ_f, the product of all the partial safety factors for the actions considered.

Material properties are normally defined by characteristic values X_k corresponding to a suitable statistical treatment of the data obtained using recognised standard test methods. Where insufficient tests have been carried out, the mean value (X_k) can be used in combination with a larger partial safety factor, γ_m. In the general case, the design characteristic value, X_d, equals X_k/γ_m. In the absence of test results, the laminae stiffness can be calculated by different named micro-mechanics methods using different interaction factors depending on the property under consideration. Some typical stiffness data are given by a weight fraction equation similar to that used in the process plant standards for chopped strand mat (CSM) and woven rovings (WR) materials.

The partial safety factor for the material is the product of the three values covering source of material properties (1.0 to 2.25), production process (1.1 to 2.7) and environmental/duration of loading (1.0 to 3.0). The more directly measured the properties, the more controlled the post-cure and the more automatic the fabrication, the lower the factors that can be used. Lower values can be used if justified by adequate control procedures.

It is also noted that the heat distortion temperature (HDT is a similar measure to T_g regarding service temperature capability) has a direct influence on the partial factor to be used. It is interesting to note the equivalence of pre-preg, filament winding and pultrusion as manufacturing processes.

Characteristic data (mean – 1.64 standard deviations) are given for several typical materials of specified lay-up and volume fraction. Creep effects are dealt with using a procedure more common for unreinforced thermoplastics, by substituting a reduced modulus that is appropriate to the loading period. Default normalised curves are given in the absence of suitable experimental data for several fibre formats. It is proposed that accelerated testing should use time–temperature superimposition.

Finally, partial safety factors are given for fatigue, depending on the level and frequency of inspection and whether a 'fail-safe' or a 'non-fail-safe' component is being considered. Values vary from 1.5 to 3. Data are also given for the maximum fatigue strain level acceptable to avoid transverse micro-cracking as a function of fatigue cycles and normal strain range, which range from 0.3% for 10^3 cycles to 0.03% for 10^7 cycles.

Interestingly, in the absence of data, equations are given that propose to predict the strength as a proportion of the characteristic strengths; however, for GRP materials there is a significant difference between the strength normally measured in a ~60 second duration test and properties obtained at the rate equivalent to the fatigue test loading rate.

Other codes and guidance documents

In the meantime, many organisations have been developing informal codes for either new build or strengthening applications using similar approaches. Some relevant documents are listed in the Further reading section.

Future trends

Progress is being made in developing the necessary infrastructure. It is most developed at the constituent and laminate level, with new product standards increasingly available. Least progress has been made at the full design code level, as in Eurocodes. It is appropriate in 2009 that these design documents should be now fully developed, approved and implemented. Two areas can be defined: new build and strengthening applications. In both cases bridge structures will have a major role, ahead of buildings. There will also be the need to continue with the approval documents currently developed by certification bodies such as Germanischer Lloyd for specialist structures such as wind turbines.

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Appendix: Other standards and codes

ISO 7370: *Glass Fibre Reinforced Thermosetting Plastics (GRP) Pipes and Fittings; Nominal Diameters, Specified Diameters and Standard Lengths.*

ISO 7432: *Glass Reinforced Thermosetting Plastics (GRP) Pipes and Fittings – Test Methods to Prove the Design of Locked Socket and Spigot Joints.*

ISO 7510: *Plastics Piping Systems – Glass Reinforced Plastics (GRP) Components.*

ISO 10467: *Plastics Piping Systems for Pressure and Non-pressure Sewerage – Glass Reinforced Thermosetting Plastics (GRP) Based on Unsaturated Polyester (UP) Resin.*

ISO 10639: *Plastics Piping Systems for Water Supply With or Without Pressure – Glass Reinforced Plastics (GRP) Based on Unsaturated) Polyester (UP) Resin.*

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ISO 14692: *Petroleum and Natural Gas Industries – GRP Piping (Four Parts)*.
EN 40-7: *Lighting Columns – Part 7: Requirements for Fibre Reinforced Polymer Composite Lighting Columns*.
EN 124: *Gully Tops and Manhole Tops for Vehicular and Pedestrian Areas. Design Requirements, Type Testing, Marking, Quality Control*.
EN 1013-2:1999. *Light Transmitting Profiled Plastics Sheeting for Single Skin Roofing. Specific Requirements and Test Methods for Sheets of Glass Fibre Reinforced Polyester Resin (GRP)*.
EN 13280: *Specification for Glass Fibre Reinforced Cisterns of One-piece and Sectional Construction, for the Storage, Above Ground, of Cold Water*.
EN 13706: *Fibre Reinforced Plastics – Specification for Pultruded Profiles*.
EN 14122: *Safety of Machinery. Permanent Means of Access to Machinery (Four Parts)*.
BS 4154: *Corrugated Plastics Translucent Sheets from Thermosetting Polyester Resin (Glass Fibre Reinforced)*.
BS 5480: *Specification for Glass Fibre Reinforced Plastics (GRP) Pipes and Fittings for Water Supply or Sewerage*.
BS 7158: *Plastics Inspection Chambers for Drains and Sewers. Specification*.
BS 7491: *Glass Fibre Reinforced Plastics Cisterns for Cold Water Storage*.
BS 7861: *Strata Reinforcement Support System Components Used in Coal Mines. Specification for Rockbolting*.
All BSI, EN and ISO standards, including copies of draft standards available for public comment, are available from BSI Customer Services, 389 Chiswick High Road, London W4 4AL (Tel +44 (0)20 8996 9001).

USA

FRP Strengthening Systems (e.g. plate bonding systems).
440.1R-06: Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars.
440.2R-02: Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.
440.2R-08: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.
440.3R-04: Guide Test Methods for Fiber-reinforced Polymers (FRPs) for Reinforcing or Strengthening Concrete Structures.
440.4R-04: Prestressing Concrete Structures with FRP Tendons.
440.5-08: Specification for Construction with Fiber-reinforced Polymer Reinforcing Bars.
440.5M-08: Specification for Construction with Fiber-reinforced Polymer Reinforcing Bars (Metric).
440.6-08: Specification for Carbon and Glass Fiber-reinforced Polymer Bar Materials for Concrete Reinforcement.
440.6M-08: Specification for Carbon and Glass Fiber-reinforced Polymer Bar Materials for Concrete Reinforcement (Metric).
440R-07: Report on Fiber-reinforced Polymer (FRP) Reinforcement for Concrete Structures.
440R-96: Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures (Reapproved 2002).
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Canada

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Canadian Standards Association. *Canadian Highway Bridge Design Code*, CSA-06-00, 2000, Toronto: Canadian Standards Association.
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Australia

Oehlers D. J., Seracino R. and Smith S. *Design Guideline for RC Structures Retrofitted with FRP and Metal Plates: Beams and Slabs*, 2007, S Publishers SAI Global Limited published under auspices of Standards Australia.

Further reading

Informal codes for either new build or strengthening applications using similar approaches.
Concrete Society. *TR55 Strengthening of Concrete Structures*. This document is based on a limit state design. Both ultimate and serviceability examples are considered for strengthening plates in glass, carbon and aramid fibre composites. The design is based on partial factors varying with fibre type (1.4 to 3.5), manufacturing route (1.1 to 2.2) and modulus of elasticity, time dependence (1.1. to 1.8). As the adhesive is critical for bonding composite plates, a partial factor of 4 is applied to the short-term strength. As well as plate reinforcements, wrapped column confinement is also considered.
Concrete Society. *TR57 Strengthening of Concrete Structures: Acceptance, Inspection and Monitoring*. Supports TR55 on additional aspects, such as inspection during installation and during the subsequent life of the structure.

Highways Agency. *Design of FRP Bridges and Highway Structures*, BD90/05. Covers bridge decking and highway structures from closed or open sections, bonded or bolted together. Material design values are lower 95% confidence limit of test data. Test loads are 1.2 × serviceability loads. Typical values of material data are given for carbon, glass and aramid, both as fibres and composites. Design life is 120 years. Serviceability limits for deflection are limited to span/300, including shear deflection. Partial factors account for transient and permanent loading as well as material creep, etc. factors.

Institution of Structural Engineers. *Interim Guidance on Design of Reinforced Concrete Structures using Fibre Composite Reinforcement*. Covers use of pultruded unprestressed reinforcements. Design based on modifications proposed to BS 8110 – Structural Use of Concrete and BS 5400: Part 4 – Code of Practice for Design of Concrete Bridges. Uses limit state design approach with appropriate partial factors.

Other similar documents include:

- FIB Bulletin 14 – similar to TR55 for concrete structures. (International Federation for Structural Concrete (FIB: Fédération Internationale du Béton)).
- CIRIA Report RP645 – similar to TR55 but for strengthening metallic structures.

Websites

Further information is available from:

- http://www.astm.org
- http://www.bsi-global.com
- http://www.iso.ch
- http://www.npl.co.uk/cog

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Chapter 57

Adhesives for externally bonded FRP reinforcement

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Externally bonded fibre-reinforced polymers (FRP) reinforcement represents one of the fastest-growing applications of polymer composites in construction. Adhesives are used to transfer shear stresses across the interface between the parent material(s) and the composite reinforcement. The principles and requirements associated with adhesive bonding are reviewed in the context of strengthening applications involving concrete, metallic and timber structures.

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Introduction

An adhesive must fulfil two fundamental requirements. First it must spread and wet out a surface; to do this, it must initially be a liquid of relatively low viscosity. Second, the adhesive must then harden to a cohesively strong solid. Hardening or curing of structural adhesives is always achieved by chemical reaction that involves polymerisation and cross-linking.

Adhesives have been developed to possess a balance of properties that enables them to adhere to a wide variety of surfaces, possess an adequate cohesive strength and appropriate mechanical properties when cured, possess good durability, and meet various application requirements in use.

The in-situ rehabilitation or upgrading of existing structures using adhesively bonded FRP represents a significant and growing activity in construction. The purpose of the adhesive is to provide a continuous bond between FRP and the parent material(s) to ensure that full composite action is developed by the transfer of shear stress across the thickness of the adhesive layer. The adhesive should be regarded as part of a *materials system* that comprises the FRP material, primer (if applicable), adhesive (or laminating resin) and parent material. A *Qualification Scheme* was developed in the Compclass project (http://www.compclass.org) that provides the materials interface with the design process for externally bonded FRP reinforcement. The website provides full details of this approach to selecting and specifying composite and adhesive materials for strengthening existing structures.

Adhesive bonding technology

The important factors involved in the formation of reliable and durable bonded joints are:

- adhesive selection;
- surface preparation;
- joint design;
- controlled fabrication;
- testing, inspection and quality control.

Structural adhesives

All structural adhesives are cross-linked because such polymers must exhibit minimal creep. Unlike thermoplastics, they do not melt or flow when heated, but lose strength and become more flexible with increasing temperature. Phenolics, epoxies, acrylics, unsaturated polyesters and some polyurethanes belong to this group, and they are all used extensively in building and construction applications (Mays and Hutchinson, 1992; IStructE, 1999; Hurley, 2000).

Epoxy resins have many attributes for use as adhesive agents in construction, namely:

- high surface activity and good wetting properties for a variety of substrates;
- may be formulated to have a long open time (the time between application and closing of the joint);
- high cured cohesive strength; joint failure may be dictated by adherend strength (particularly with concrete substrates);
- may be toughened, by the inclusion of a dispersed rubbery phase, to improve impact and low-temperature performance;
- lack of by-products from curing reaction minimises shrinkage and allows the bonding of large areas with only contact pressure;
- low shrinkage compared with polyesters, acrylics and vinyl types; hence, residual bondline strain in cured joints is reduced;
- low creep and superior strength retention under sustained load with most formulations;
- can be made thixotropic for application to vertical surfaces (i.e. a shearing force is required to make the adhesive flow);
- able to accommodate irregular or thick bondlines (e.g. concrete adherends);

- may be modified by: (a) selection of base resin and hardener; (b) addition of other polymers; (c) addition of surfactants, fillers and other modifiers;
- normally suitable for service operating temperatures in the range −30°C to +60°C. A higher upper limit may be achieved by using warm- or heat-cured formulations that possess a higher glass transition temperature, T_g.

Two-part cold-curing paste epoxy adhesives that have been specially developed for use in construction are appropriate for FRP strengthening. The principal requirements for bonding composite materials systems are given by Hutchinson and Quinn (1999).

Adhesion and surface preparation

Adhesives join materials primarily by attaching to their surfaces. Being liquid, adhesives flow over and into the irregularities of a solid surface, coming into contact with it and, as a result, interacting with its molecular forces. The adhesive then solidifies to form the joint.

Surface preparation of the adherends is one of the most important aspects in adhesive bonding. It is a critical topic that is not often given the attention by designers, operatives and their supervisors that it demands. Adhesives are frequently blamed for 'not sticking', but the source of the trouble generally lies with the surface preparation. Some particular lessons that have been learned over many years of structural adhesive bonding in construction are as follows:

- Most adherends need to be prepared properly, **especially to secure long-term bond durability**. Metals and thermoplastics present the greatest challenges.
- Some metals and FRP materials can be procured in a pretreated and pre-primed state.
- Greater use of primers and coupling agents formulated specifically for structural adhesive bonding should be used in conjunction with metallic adherends.
- Minor changes in preparation techniques can result in significant differences in the surface condition of materials such as metals. For metal treatment, for example, a change to grinding from gritblasting could lead to bonded joints of significantly lower strength and durability.
- Treated surfaces must be bonded immediately, or coated with a primer or chemical coupling agent.

Primers and coupling agents

Primers, being low-viscosity materials when compared with most adhesives, assist adhesion either by partially penetrating the pores of a porous surface or by forming a chemical link between the surface and a relatively high viscosity adhesive. Adhesive primers are important for metallic substrates and for porous surfaces such as concrete and masonry. In structural adhesive bonding, any primers used must possess cohesive strength.

The alternative approach to primers utilises a very thin layer of a chemically reactive system based on silane coupling agents, applied as a dilute solution (0.1–5%). These coupling agents can work well on metals but care in the use of such systems is required for use on site.

Behaviour of joints and design principles

Many aspects must be considered in the design of a structural bonded joint in which the load is carried by the adhesive only. Joint behaviour is influenced by the nature of the applied loading, the geometry of the bonded area, the bulk and surface properties of the adherends, and the properties of the adhesive itself. The guiding principles that should be adopted by all designers of bonded joints are to:

- provide the maximum bond area;
- stress the adhesive in the direction of maximum strength (i.e. in shear or compression), and avoid the build-up of secondary tensile stresses;
- avoid large stress concentrations which are associated with relatively high modulus adhesives and flexible adherends;
- make the adhesive layer as uniform, void-free and as thin as practically possible;
- maintain a continuous bondline.

Peel stresses are particularly damaging because adhesive bonds possess less resistance to peel than to any other mode of loading. Peel stresses can also lead to premature failure or delamination of materials such as concrete, timber, FRP and wrought iron. General guidance on engineering bonded joint design is given by Adams *et al.* (1997). Design guidance on joining composite materials is given by Clarke (1996) and Hutchinson (1997).

Fabrication of joints

Satisfactory fabrication relies upon the provision of suitably qualified materials, detailed designs, and appropriate method statements containing fabrication processes. Training of site operatives and their supervisors is essential to ensure quality of workmanship (CSWIP, 2006). The main topics for consideration are:

- storage of materials, including solvents, primers and adhesives;
- protection of the working environment;
- control of adherend moisture contents (e.g. concrete, timber and FRP);
- surface preparation of adherends;
- priming of adherends (if applicable), including considerations of timing;
- adhesive mixing, dispensing and application;
- component fit-up, including temporary works and fixtures;

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- control of bondline thickness and adhesive fillets at joint edges;
- curing;
- overcoating and painting.

Detailed commentaries on these aspects are provided in a variety of sources (e.g. Mays and Hutchinson, 1992; Adams *et al.*, 1997; IStructE, 1999).

Health and safety

Adhesives, primers and solvents will have quoted threshold limit values (TLVs) for short- and long-term exposures. The manufacturers' health and safety data sheets must always be consulted before use: they should be used as the basis of any COSHH (Control of Substances Hazardous to Health) assessment. Adhesives should be used in accordance with the manufacturers' recommendations, taking the necessary precautions such as the use of protective clothing during mixing and application. For example, epoxy adhesives can result in skin irritation if direct contact is made. Similarly, when preparing surfaces, provision must be made for adequate extraction of noxious fumes, dust, etc. and a suitable supply of fresh air.

Surface preparation

The purpose of surface preparation is to remove contamination and weak surface layers. It is not a prerequisite of adhesion that the surface actually needs to be rough. However, the generation of micro-roughness ($<0.1\,\mu m$) is beneficial, as is the introduction of new chemical groups onto the surface to link with the adhesive (or primer). The key stages involved in achieving this purpose are:

- cleaning;
- material removal and surface modification;
- further cleaning (to remove contamination introduced by treatments, such as oil-mist, dust or chemical residues).

It should be noted that solvent wiping alone may simply spread oils and greases without removing them from the surface.

For most strengthening applications the adherend material(s) must be treated in situ, sometimes under difficult conditions. The plane of the surface(s) to be treated (horizontal, vertical, overhead, etc.) has a large bearing on the selection of an appropriate method of surface preparation. The choice of method, or combinations of methods, depends upon the costs, the scale and location of the operation, access to equipment and materials, and health and safety conditions.

A summary of suitable preparation techniques for different substrates is given in **Table 1**.

ISO 8504-1: 1992 and NPL (2000) provide guidance on the preparation methods that are available for cleaning metallic substrates and ISO 8501-1: 1988 incorporates the well-established Sa-series for blast cleaning.

Material	Procedures
Steels and iron	Grind off major surface irregularities Solvent cleaning Dry gritblasting Removal of dust Application of a primer, if necessary
Concrete	Remove any damaged or substandard concrete and replace with good-quality material Remove laitance, preferably by gritblasting Remove dust and debris by brushing, air-blast or vacuum Drying of the surface, if necessary Application of a primer, if necessary
Timber	Cutting with a plane, saw, auger, chisel or similar sharp tool Removal of dust Localised drying to <20%, if necessary Application of a primer, if necessary
FRP (thermoset matrix)	Solvent cleaning Light abrasion and solvent cleaning *or* removal of a peel ply layer

Table 1 Suitable preparation techniques

Guidance on the preparation of concrete substrates is provided by ASTM D4258 (1983), and the Concrete Society (2004).

River *et al.* (1991) and Davis (1997) provide good accounts of the nature of timber surfaces and their treatment.

ASTM D2903 (1984) provides a commentary on preparation methods for plastics and composites.

Pull-off tests to EN 1542 or ISO 4624: 2002 can demonstrate that the adhesive bond is either of sufficient strength or at least stronger than the parent material or other components of the materials system.

Strengthening of concrete, metallic and timber structures using pre-cured composites

Many different forms of FRP (pultruded plates, bars and rods, pre-cured shells) have been developed to meet the requirements of strengthening a variety of structures comprising concrete, masonry, metals and timber.

Epoxy adhesives have been used for the bonding of steel plate reinforcement to the external surfaces of concrete structures for strengthening purposes since the late 1960s. The 1990s witnessed an increased use of carbon-fibre-reinforced plastic (CFRP) to replace steel because of advantages associated with improved installation and potentially enhanced long-term durability of the connection. The CFRP can even be prestressed prior to bonding.

The design stage for flexural strengthening is characterised by four key steps:

- material selection, including adhesive system and composite reinforcement;
- material property determination required for design (by testing);
- design calculations to determine the required cross-sectional area of reinforcement and acceptable stresses in the adhesive layer;
- design check to confirm adequate strength of reinforcement and adequate adhesion to the substrates.

This final step links the *Compclass Qualification Scheme* to the design process.

Concrete structures

In buildings it is sometimes desirable to remove supporting walls or individual supports, leading to the need for local strengthening. A common requirement is the need to strengthen the floor–beam connection in order to increase the capacity for increased loading, or to compensate for inadequate original design by facilitating alternative load paths. Examples include the installation of lift shafts in existing buildings, adding further floors to office blocks and hospitals, enhancing the integrity of power station structures and limiting the deflection of slabs in multi-storey car parks. With bridges and culverts, strengthening may become necessary as a result of damage, deterioration or design code changes. In most cases the materials involved are flat pultrusions bonded to concrete surfaces (**Figure 1**), but bars may also be inserted into pre-cut slots (**Figure 2**). Bridge deck (and other) columns can be wrapped with reinforcement to preserve integrity, to mitigate the effects of impact damage and to reduce seismic damage. These applications can involve pre-cured shells and pultruded sheets, site-laminated fabric materials and automated winding techniques. With slabs, strengthening may be in flexure, shear, or both; with columns, the intention is confinement. Relevant design guidance is given by Highway Agency (1994), FIB (2001), ACI 440 (2002) and Concrete Society (2004).

Figure 2 CFRP bars bonded into slots cut in concrete cantilever deck (courtesy of Tony Gee and Partners)

Metallic structures

The main applications to date have involved cast and wrought iron structures, often of historical importance. Pultrusions are easily attached to the relatively flat surfaces of beams (**Figure 3**) while fabrics are more suited to curved members and intricate shapes. The purpose of strengthening may be to reduce the effective stresses in the original structure caused by loading or thermal effects, and the former represents an innovative use of prestressed CFRP pultrusions in Oxford, UK (Darby *et al.*, 1999). Relevant design guidance is given by Moy (2001) and CIRIA C595 (2004).

Figure 1 UD CFRP plates bonded to top surface of concrete around columns (courtesy of http://www.compclass.org)

Figure 3 UD CFRP plates bonded to cast iron girders (courtesy of http://www.compclass.org)

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Figure 4 GFRP pultruded bars bonded into timber to make a new beam-end connection (courtesy of Rotafix Ltd)

Timber structures

Common resin-bonded methods of repair involve the removal and reinstatement of beam-ends, parts of trussed rafters, and the ends of columns. Combinations of resin grouts, steel bars, pultruded FRP rods and structural adhesives are used. Strengthening of beams may involve the attachment, or insertion into slots, of pultruded FRP bars or plates using a structural adhesive (**Figure 4**). Many hardwood buildings and bridges of historical importance have been repaired and strengthened with minimum intervention (see http://www.licons.org). The techniques have been so successful and aesthetically pleasing (i.e. invisible) that there is much interest in making reinforced bonded connections in new build and in enhancing the flexural performance of glued laminated timber beams significantly by including FRP laminations in the section. Design guidance is provided by STEP 1/2 (1995), TRADA (1995) and www.licons.org.

General application considerations

Application of the composite materials system must be undertaken shortly after surface preparation.

The composite material should be checked for dimensions, mechanical properties, straightness and conformity to the specification. The geometry of the plate should also be checked against the substrate to which it is bonded and any gaps should be filled with appropriate material.

Mixing and application of the resin and hardener should be according to the specification or adhesive supplier's instructions. The amounts of adhesive being mixed should not exceed the maximum recommended because larger volumes may reduce the efficiency of the mixing process or increase the temperature of the mixed resin, reducing its pot life.

The temperature will influence the application and ease of spreading of the adhesive. However, ease of application is often inversely proportional to the degree of grab. Many applications are on vertical or the underside of horizontal surfaces where the grab of the adhesive layer is important for positioning of the composite plate. If the viscosity of the mixed resin is too low, then the grab of the adhesive may be insufficient for alignment of plates. Constant, even pressure should be applied to plates such that any excess adhesive is pushed to the edges and a uniform adhesive layer is achieved.

The cure of an adhesive is strongly influenced by temperature and the correct mixing of resin constituents. The work area can be protected and heated if necessary, and most adhesive systems benefit from warm temperature curing, say between 30°C and 40°C, to promote good adhesion, accelerate cure development and to achieve a higher T_g. This is important for metallic substrates.

QA and testing

A summary of the recommended quality assurance (QA) tests for the application of adhesives to externally bonded reinforcement is presented in **Table 2** (http://www.compclass.org). Off-site testing should be performed by an experienced independent test house. Where testing involves a bonded joint, it is essential to record the locus of failure as well as the failure load.

Measurement location	Property	Test method	Number of tests (per batch)	Comments
On-site (adhesive)	Hardness (state of cure)	ISO 868: 2003	At least 10 tests, evenly distributed over the composite	On-site hand-held test method (Shore D hardness) to assess degree of cure
	Adhesion (parent and FRP reinforcement materials)	EN 1542: 1999	At least 5 tests	Pull-off tests using steel dollies: 25 mm diameter for metals, 50 mm diameter for concrete
Sample prepared on-site, measured off-site	Glass transition temperature (adhesive)	EN 12614 or ISO 11357-2: 1997 or ISO 6721: 2002	At least 3 tests	Same method for T_g measurement should be used throughout the qualification scheme
	Tensile modulus and strength (adhesive)	ISO 527:1997	At least 3 tests	Dumbbell specimen
	Adhesion (adhesive and composite)	Lap shear joint performance ASTM D 5868	At least 3 tests	Ideal thickness of composite: <5 mm Ideal bondline thickness ~0.5 mm

Table 2 Summary of QA tests for pre-cured composites applied on site

Strengthening of structures using in-situ lamination techniques

There are various methods for on-site laminate application. The most common involves reinforcement and resin supplied separately, requiring cutting and metering on a bespoke basis. Another is pre-packaged material supplied as a prepreg roll. Both alternatives are applied using contact moulding. Another variant is for the reinforcement to be supplied pre-cut and then applied on-site using a resin infusion process. Where a separate adhesive material is required to achieve a bond between individual laminate layers, the adhesive must be applied uniformly. Consolidation of the layers of reinforcement and resin can be accomplished through the use of simple hand tools such as rollers. This must be carried out thoroughly to ensure adhesion between individual layers and to remove air inclusions; excessive pressure should be avoided to prevent damage or undesired reorientation of fibres. Curing considerations are similar to those described earlier.

Summary and future trends

FRP strengthening systems are very versatile and the range of applications is enormous. If correctly designed, specified and installed, FRP systems should be durable with an adequate service life. However, the techniques are highly susceptible to the quality of workmanship and so education and training of the operatives and their supervisors is essential.

Control of adhesive bonding operations is crucial to the satisfactory fabrication of all reliable and durable bonded joints. There are many aspects that must be considered including the storage of materials, protection of the working environment, surface preparation of the components, adhesive mixing, dispensing and application, bondline thickness control and curing. When bonding metallic substrates, one of the most difficult aspects to control on site is surface preparation.

The *qualification scheme* approach developed in the Compclass project enables designers to select composite materials systems (including adhesives and primers) on the basis of performance requirements. Surface preparation and appropriate test methods for assessment are central to this scheme.

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Chapter 58

Applications of fibre-reinforced polymer composite materials

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Fibre-reinforced polymer (FRP) composite materials have shown that they are able to provide structurally efficient and competitive alternatives to the more conventional construction materials when utilised for the rehabilitation of concrete, steel, timber and masonry structural members, for bridge deck construction, and for the retrofit of columns in seismic regions. In these applications the attributes of low weight, tailorability performance characteristics based on designed anisotropy, corrosion resistance and lightweight are significant advantages. The most significant growth area currently is in the rehabilitation of structural members using externally bonded composites but there is also immense potential for the development of a combination of FRP composites with the more conventional construction materials. This combination not only builds on the inherent advantages of FRP composites but also provides for optimum integrated usage of FRP composites and (say) concrete in their most efficient modes of tension and compression, respectively. Other materials could be combined with composites to provide a more economical use of both materials This chapter is divided into a number of sections concerned with the applications of advanced polymer composites over the past 35 years of the building industry and the past 20 years of the civil engineering industry. The reasons for using the polymer composite and construction systems in the various examples given are discussed and further reading material is indicated.

Introduction

Polymer composites were first used in the building industry during the 1970s as non-load-bearing and semi-load-bearing infill panels manufactured by the hand lay-up process; the polyester polymer was used. During the early part of this period little thought was given to the long-term durability of the material; the main concern of the fabricators was to find new outlets for their products and new methods of manufacture of large panels for use in the building industry. Consequently, some additives to improve certain physical properties were incorporated into the polymers without a full understanding of their effect on the durability of the material or indeed were omitted in cases where additives should have been added to resist forms of degradation. Examples can be cited where the material 'yellowed' after a period of exposure to sunlight because ultraviolet stabilisers had not been incorporated into the polymer at the time of fabrication. Another serious fault that occurred at that time was laminate leaching. Leaching occurs after exposure of the laminate to the atmospheric weather and is characterised by a loss of resin from the laminate, leaving the glass fibres exposed to attack by moisture. This problem is generally caused by an insufficient curing of the resin. Eventually these problems were understood and overcome and the polymers were improved in quality. The advanced polymer composites did not enter the civil engineering construction industry until the late 1980s when vinylester and epoxy polymers were mainly used in place of polyester. In addition, universities, research institutes and industrial firms were involved in researching the in-service and mechanical properties, the design and the testing of structural units manufactured from polymer composites, this was followed by the involvement of interested civil engineering consultants undertaking industrial research and utilisation of the material in practice. The application of advanced polymer composite, over the past 35 years of the building industry and the past 20 years of the civil engineering industry, can be conveniently divided into specific areas, which will be discussed briefly, as follows:

i Building industry

- Infill panels and new building structures.

ii Civil engineering industry

- Civil engineering structures, fabricated entirely from advanced polymer composite material.
- FRP–concrete beam construction.
- Internal reinforcement to concrete members.
- External reinforcement for metallic and reinforced (RC) structures (civil engineering structural systems) including seismic retrofit of RC and masonry structures.
- Replacement of degraded concrete bridge deck systems (civil engineering structural systems).

- FRP confining of concrete columns.
- Bridge enclosures.

In addition, polymers are utilised in the manufacture of bridge bearings and vibration absorbers.

Ideally when new civil engineering structures are manufactured from polymer composite systems the component parts should be modular and their assembly rapid and simple. Furthermore, any construction or long maintenance period of the highway infrastructure causes disruption to the flow of traffic and is extremely expensive. Consequently the structural material used on highway construction should be durable and any connections must be reliable. As FRP composite materials are durable and lightweight, they fulfil these requirements provided the initial design of the basic building modular system is properly undertaken and the material properly installed.

The building industry

Mondial House erected on the north bank of the Thames in London 1974

The main development in the building industry took place in the 1970s. One of the prestigious semi-loading buildings to be erected in 1974 was Mondial House; it was demolished in 2007 to allow for redevelopment of that area. The building was clad above the upper ground-floor level and the panels were manufactured from glass-fibre polyester resin. The outer skin of the panel included a gel coat which used isophthalic resin, pigmented white, with an ultraviolet stabiliser backed up with a glass-fibre-reinforced polymer (GFRP) laminate; the latter used a 3 oz per square foot chopped strand mat and a self-extinguishing laminating resin reinforced with 9 oz per square foot glass-fibre chopped strand mat reinforcement. Some degree of rigidity was obtained from a core material of rigid polyurethane foam bonded to the outer skin and covered on the back with a further glass-reinforced laminate; this construction also provided thermal insulation. Further strength and rigidity were obtained by the use of lightweight top-hat section beams, manufactured as thin formers, and incorporated and over-laminated into the moulding as manufacture proceeded. The effect of the beams was transferred to the front of the panel by means of glass-fibre-reinforced ties or bridges formed between the polyurethane foam at the base of each beam. The face of the beam was reeded on the vertical surfaces in order to mask any minor undulations and to provide channels off which the water ran and thereby cleaned the surface. The reeding also gave the effect of a matt panel without reducing the high surface white finish. The structure was visually inspected in 1994 by Scott Bader and the University of Surrey and the degradation was found to be minimal. In 2007 a part of the composite material from the demolished structure was analysed at the University of Surrey for any variations in the mechanical properties (Sriramula and Chryssanthopoulos, 2009).

An 'all polymer composite' classroom of primary school, Thornton Clevelys, Lancashire, UK, 1974

The classroom is an 'all composite' FRP building in the form of a geometrically modified icoshedron and is manufactured from 35 independent self-supported tetrahedral panels of chopped strand glass-fibre-reinforced polyester composite. Twenty-eight panels have a solid single skin GFRP composite skin and in five of these panels circular apertures were constructed to contain ventilation fans. In the remaining seven panels, non-opening triangular windows were inserted. The hand lay-up method (the technique has been modified over the years and is now known as the wet lay-up method) was utilised to manufacture the E-glass fibre/polyester composite skins. The inside of the panels have a 50 mm thick integral skin phenolic foam core acting as a non-load-bearing fire protection lining to the GFRP composite skins. The icoshedron structure is separated from the concrete base by a timber hardwood ring. The fibre-reinforced polymer (FRP) panels were fabricated on to a mould lining of perspex with an appropriate profile to give a fluted finish to the flat surfaces of the panels. The edges of the panels were specially shaped to provide a flanged joint which formed the connection with adjacent panels. Sandwiched between two adjacent flanges is a shaped hardwood batten, which provides the correct geometric angle between the panels; the whole is bolted together using galvanised steel bolts placed at 450 mm intervals. The external joint surfaces between the adjacent panels were sealed with polysulphide mastic. The glass windows were fixed in position on site by means of neoprene gaskets.

When the classroom structure was under construction in 1974, a fire test at the Building Research Establishment (BRE) Fire Research Station at Thornton Cleveleys, Lancashire, UK was undertaken on four connected GFRP panels, with the integral skin phenolic foam in place. At the same time, tests were also undertaken on an identical geometrically shaped school system used at that time. The results demonstrated that the GFRP classroom had over 30 minutes fire rating whereas the existing school system had only 20 minutes.

The civil engineering industry

'All polymer composite' structures

For the civil engineering industry the structural systems which are manufactured entirely from advanced polymer composites may be divided into two basic groups:

1 The substitution of the traditional materials with new FRP materials.

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2 The new material-adapted concept.

These systems, like those of the building industry produced to date, have tended to be single prestigious structures, manufactured from 'building blocks'; the advantages of the 'building block' construction may be stated as:

a Construction of identical units manufactured on a production line.
b Controlled mechanised/manual factory manufacture and fabrication of identical structural units. The transportation to site of the lightweight units should present no particular problem with units that can be readily stacked. Furthermore, it is more economical to transport lightweight stacked FRP units than the heavier steel and concrete units.

Examples of some of these UK structures are the Aberfeldy Footbridge, Scotland, the Bonds Mill Single Bascule Lift Road Bridge, Oxfordshire, the road bridge over the River Cole at West Mill, Oxfordshire and the Willcott Bridge, A5 Nescliffe bypass, Gloucestershire.

The three former structures were of modular construction manufactured utilising advanced composite materials. For the construction to be successful the material has to be durable and the components should be modular in form. The assembly of the units should be rapid, simple and have reliable connections. Advanced polymer composite materials are durable and lightweight; consequently, they fulfil these requirements provided the initial design of the basic building modular system is properly undertaken and the material properly installed.

The *Aberfeldy Footbridge* was erected in the early 1990s. The footbridge is of cable-stay construction and the composite material used in the deck and the tower supports of this bridge was GFRP and, although this material is strong in tension, it has a low modulus of elasticity value (see Chapter 52). This low stiffness was increased by geometrically shaping the modular system to give it an enhanced *EI* value. The first major UK, all-composite, versatile automated modular system was designed and produced by Maunsell Structural Plastics (now Faber Maunsell), Beckenham, UK. The modular system was introduced into the construction industry as the advanced composite construction system (ACCS) (Hollaway and Head, 2001) and was first used in modular form as the bridge enclosure to the A19 Tees Viaduct at Middlesborough. The ACCS modular system relies upon the cold-cure adhesive bonding with epoxy adhesive. This project clearly showed the feasibility and potential for advanced composite bridges and constituted the pioneering work in this field; however, this bridge was a demonstration project. The system was initially manufactured in the UK by GEC Plastics (now Fibreforce) by the pultrusion technique using isophthalic polyester resin and unidirectional, bidirectional and chopped strand mat glass fibre reinforcement for the main structural members. Strongwell, Bristol, VA and Chatfield, MN, USA, now hold the manufacturing licence for the plank and produce similar panels under the trade name of COMPOSOLITE®. The design method used for the ACCS modules for the Aberfeldy Footbridge was based upon the limit state design principles. This principle provided a logical design procedure which identified the limit state at which the structure ceased to fulfil its design functions.

Figure 1 Single bascule lift bridge – the Bonds Mill Bridge (courtesy of Faber Maunsell)

The *Bonds Mill Bridge* is a single bascule lift bridge and is shown in **Figure 1**. It consists of two epoxy bonded ACCS multicell box beams. The 80 mm × 80 mm × 9 m long compressive cells and the web cells of the ACCS planks forming the box beams have been infilled with slow foaming epoxy foam of density 90 kg/m^3 (manufactured by CIBA Polymers). This material provides uniform support to the thin walls of the ACCS units, allowing load transfer, without high local bending stresses. The weight of the 9 m span bridge is 4.5 t. The running surface of the polymer composite bridge is made from ACME panels (a proprietary system of epoxy-coated panels with grit bonded/embedded into it) which are bolted onto the top flange of the GFRP box beam.

Willcott Bridge is a 51.3 m single-span suspension footbridge with a slightly cambered slender deck providing a footway 2 m wide; **Figure 2** is a photograph of the bridge. The cross-section of the deck of the bridge consists of six GFRP 'Maunsell Planks' (Hollaway and Head, 2001); three planks are placed in the top surface of the deck and three in the bottom. These planks are connected by four webs made from two three-way connectors. The edge beams (outer webs) of the deck are assembled using five three-way connectors. The whole deck system is constructed using the ACCS planks, three-way connectors, toggles and adhesive to join the 'planks' longitudinally. The units comprising the deck were manufactured by

Figure 2 Willcott Bridge – single-span suspension footbridge (courtesy of Faber Maunsell)

Strongwell Corporation, USA; this firm now markets the ACCS system as Composolite. To increase the mass of the deck, the cells of the middle planks are filled with ballast, which were made from mortar blocks; these were sized to enable them to easily slide into the cells. The surfacing of the deck is provided by interlocking rubber blocks, manufactured from recycled vehicle tyres. Resin block inserts were bonded to the sides of the outer edge beams thus facilitating the joining of the hangers and parapet posts. The construction of the bridge deck was made in three lengths each measuring about 17 m; this method was used to facilitate transportation of the deck sections to site. These separate lengths of deck were joined by mechanical and adhesive connections; these were staggered in both vertical and lateral directions.

There have been a number of footbridges recently built that have used the ACCS 'Maunsell Plank'; for instance, the New Chamberlain Bridge, Bridgetown, Barbados, was completed in 2006. It spans the Careenage River, was designed by Faber Manusell and was constructed from Composolite. This bridge replaced the steel horizontal swing bridge that had been built in 1872. The composite bridge is composed of two 2.13 m (7 ft) raised sidewalks with handrails and a 4 m (13 ft) wide road in the centre, which is for emergency vehicular traffic only; the lift bridge is of similar construction to that of the Bonds Mill Bridge. Another ACCS 'Maunsell Plank' footbridge erected in 2007 is the Network Rail Footbridge which crosses the Paddington–Penzance rail at St Austell, UK. It has a span of 14 m and was constructed from Composolite; the consultant for this bridge was Parsons Brinkerhoff, UK.

Bridge enclosures and fairings

The need for regular inspection and maintenance of bridge structures is causing increasing concern because of the disruption caused to travellers if closure is required to maintain a bridge. Furthermore, the cost of the closure will be extremely large. Moreover, stringent standards are increasing costs of maintenance work over or beside busy roads and railways. Most bridges designed and built over the last 30 years do not have good access for inspection and in northern Europe and North America deterioration caused by de-icing salts is creating an increasing maintenance workload (see External reinforcement to reinforced concrete structural members).

The concept of 'bridge enclosure' was developed jointly by the Transport Research Laboratory (TRL, formerly TRRL) and Maunsell, Beckenham, UK (now Faber Maunsell) in 1982, to provide a solution to the problems. The function of these enclosures is to erect a 'floor' underneath the girder of a steel composite bridge to provide inspection and maintenance access. In addition to providing these structural requirements, enclosures allow greater freedom of aesthetic expression independent of the strength requirements. The floor is sealed onto the underside of the edge girders to enclose the steelwork and protect it from further corrosion. It has been shown (McKenzie, 1991, 1993) that once the enclosures are erected the rate of corrosion of uncoated steel in the protected environment within the enclosure is 2–10% that of painted steel in the open. Although this enclosure space has a high humidity, chloride and sulphur pollutants are excluded by seals, so when condensation does occur (as in steel girders) the water drops onto the enclosure floor which is set below the level of the steel girders and there it escapes through small drainage holes. The floor and fixings are non-corrosive and no water is able to pond against the steel and hence corrosion is prevented.

Polymer composites are ideal materials from which to manufacture enclosure floors because they add little weight to the bridge and are highly durable, particularly as the polymer composite, being under the bridge soffit, is protected from direct sunlight. Consequently, most bridge enclosures which have been erected in the UK, have utilised polymer composites. The first major example which utilised this technique was in 1988–1989 when the A19 Tees Viaduct at Middlesborough was fitted with the Maunsell 'caretaker' system.

Since 1994 health and safety legislation and design guidance published by the Highways Agency has had an impact on the design of bridge enclosures. Currently their designs are covered by BD 67/96 (Highways Agency, 1996). The requirement for wind loading is covered by BD 37/01 (Highways Agency, 2001). In addition, when enclosures are placed under railway bridges, aerodynamic pressure caused by the displacement of air due to the passage of a train is significant and, therefore, the allowable deflection, and the design of the fixings for the enclosure must be carefully considered. **Figure 3** illustrates the bridge enclosure on the approach span of the Dartford River Bridge (QE2) where it goes over the Channel

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Figure 3 The bridge enclosure on the approach span of the Dartford River Bridge (QE2) (courtesy of Faber Maunsell)

Tunnel Rail Link (CTRL) (before the train rails were installed).

FRP–concrete beam construction

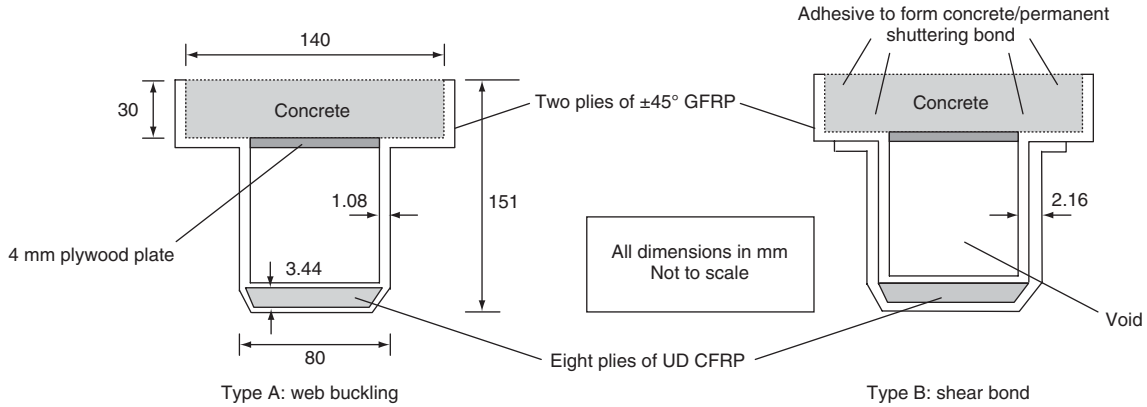
The combination of advanced polymer composites (APC) and a conventional civil engineering material, i.e. concrete, to form a 'duplex' beam has been researched by Triantafillou and Meier (1992). The emphasis of their work was to use the concrete in the compressive and APCs in the tensile regions of a beam. Thus the results in the two materials are used to their best structural advantage. During the early to mid-2000s, Hulatt *et al.* (2003a,b, 2004) developed the idea of the duplex beam further by testing and numerically analysing the system under various geometries and loading configurations. The APC materials used in this work were prepregs of glass and carbon supplied by Advanced Composites Group (ACG), Heanor, UK; this material has been described in this chapter. **Figure 4** shows a digramatic elevation of the FRP–concrete beam. As a result of the above work, NECSO Entrecanales Cubiertas, Madrid, Spain and ACG Ltd, Heanor, UK have developed a 'duplex' beam element. This utilises the high compressive strength of concrete and the high tensile strength of the carbon fibre. Load testing at 80% of ultimate load demonstrated that the duplex beam behaved as a typical steel girder, indicating that the traditional principles of flexural design can be utilised. **Figure 5** shows the completed bridge. Analysis indicated that the manufacturing cost of a duplex beam is comparable to long-span concrete beams. The real benefit is in the significant cost savings provided due to the lower weight and reduction through life cost of the beam. Natural opportunities for this technology are more site installations and refurbishment of infrastructure in developing countries or war-damaged regions.

External reinforcement to reinforced concrete structural members

a Upgrading reinforced concrete members
b Seismic retrofit of RC columns
c Upgrading steel structural members.

The cost of repair, upgrading and strengthening/stiffening of deteriorated, damaged and substandard infrastructure has become one of the fastest growing and important challenges confronting the bridge engineer worldwide. It is generally much less expensive and less time consuming to repair a bridge or building structure than to replace it.

Reinforced-concrete bridge structures, for a variety of reasons, may be found to be unsatisfactory. In the design



Materials
The APC materials used in this experimental project were prepregs of glass and of carbon supplied by Advanced Composites Group (ACG), Heanor, UK. The materials were compacted under a vacuum-assisted pressure of 1 bar and cured at an elevated temperature of 60°C applied for 16 h or 80°C applied for 4 h

Figure 4 Experimental duplex beam

Figure 5 The completed NECSO Bridge under site loading (Entrecanales Cubiertas, Madrid, Spain and ACG Ltd, Heanor, UK developed 'duplex' bridge beam) (courtesy of ACG Derbyshire, UK)

and construction phase, causes of deficiency include marginal design/design errors causing inadequate factors of safety, the use of inferior materials, or poor construction workmanship/management, causing the design strengths not to be achieved. In service, changes in use of a structure include:

- *Increase live load* – changes in use of a building resulting in greater imposed live loads, increased traffic on a bridge.
- *Increase dead loads* – additional load on the structure due to new construction.
- *Increased dead load and live load* – widening a bridge to provide for an extra lane of traffic.
- *Modern design practice* – changes in design techniques and updates of codes of practice.
- *Material deterioration* – corrosion of the internal reinforcement, particularly in marine or industrial environments, carbonation of the concrete or alkali–silica reaction.
- *Prestressed concrete beams* – strengthening measures may be required to prevent further loss of prestress.

Upgrading reinforced-concrete members by plate bonding

Initially unstressed FRP plates

Bonding FRP composites. Many experimental and analytical research investigations have been undertaken on reinforced-concrete beams strengthened by FRP composites, some of these are discussed in Triantafillou and Plevris (1991), Hollaway and Leeming (1999), Teng *et al.* (2002), The Concrete Society (2000, 2003), Oehlers and Seracino (2004).

The FRP composite plate material used for the bonding operation is generally the high-modulus CFRP, AFRP (Kevlar 49) or GFRP composites and these composites will be fabricated by one of three methods, namely:

1. The pultrusion technique, in which the factory made rigid precast FRP plate is bonded onto the degraded member with cold-cure adhesive polymer.
2. The factory-made rigid fully cured FRP prepreg plate bonded to the degraded member with cold-cure adhesive polymer.
3. The hot-melt FRP prepreg/adhesive film placed onto the structural member and both components are cured simultaneously on site under pressure and elevated temperature (see chapter on Advanced polymer composites).
4. The wet lay-up process (see chapter on Advanced polymer composites).

Figure 6 illustrates the upgrading of a RC beam utilising FRP composites. Method 3 is superior to the precast plate and cold-cure adhesive systems as the site compaction and cure procedure of the prepreg and film adhesive ensures a low void ratio in the composite and an excellent join to the concrete. The drawback to this method, currently, is the cost; it is about twice as expensive as methods 1 and 2. The preferred manufacturing system currently for upgrading is either method 1 or 2. This implies that:

1. The plate material cannot be reformed to cope with any irregular geometries of the structural member.
2. A two-part, cold-cure epoxy adhesive is used to bond the plate on to the substrate.

The cold-cured adhesive is the Achilles' heel of the system particularly if it is cured at a low ambient temperature, and without post-cure, see Chapter 51 (Polymerisation) and Chapter 52 (Durability).

Power actuated (PA) fastening 'pins' for fastening FRP composites. A viable alternative to the adhesive bonding of a preformed pultruded or a prepreg rigid plate is by mechanically fastening the FRP plate to the RC beam. The fastening is achieved by using many closely spaced steel power actuated (PA) fastening 'pins' and a limited number of steel expansion anchors. The method, known as the mechanically fastened unbonded FRP (MF-UFRP) method, is rapid and uses conventional handtools, lightweight materials and unskilled labour. In addition, the MF-UFRP method requires minimal surface preparation of the concrete and permits immediate use of the strengthened structure. The advantage of using multiple small fasteners as opposed to large-diameter bolts, which are generally used for anchorages, is that the load distributes uniformly over the FRP strip and this reduces the stress concentrations which can lead to premature failure. The method has been developed by researchers at the University of Wisconsin, Madison, USA (Bank, 2004). An analytical

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analysis to predict the strength of concrete beams upgraded with the MF-FRP method has been discussed by Bank and Arora (2007) and Arora (2003). Bank *et al.* (2003a,b) have discussed the strengthening of a 1930 RC flat-slab bridge of span 7.3 m by mechanically fastening the rigid FRP plates using the MF-FRP method.

Ki (2005) compares the experimental testing of flexural strengthened beams with CFRP/GFRP custom strips using the alternative methods of joining the FRP plates to the RC beams by adhesive bonding and by MF-UFRP. The custom strips (trademarked SafStrip) were manufactured by Strongwell, USA using the pultrusion technique. The strips used a combination of unidirectional carbon fibre, E-glass roving, E-glass continuous strand mat and a vinlyester resin matrix. The results showed that the mechanically fastened strips had an increase of 10.45% in the yield and ultimate moments over the unstrengthened control beam. This method indicated comparable strengthening to the plate bonding method and because of the fastening method used resulted in strengthened beams with a ductile failure mode; the plate bonding method gave brittle failure modes. It was noted that the FRP strip attachment with the MF-UFRP was completed in a shorter time period compared with the bonding.

El-Maaddawy and Soudki (2007) examined the potential use of mechanically anchored unbonded FRP (MA-UFRP) to upgrade RC slabs deficient in flexural strength. The experimental results of these upgraded slabs were compared with those that had been strengthened with externally bonded preformed FRP (EB-FRP) plates. They showed that the MA-UFRP strengthening system was effective in increasing the slab strength but the strength gain was less than that obtained by the use of the EB-FRP strengthening system. The MA-UFRP system resulted in an average strength gain of 33% relative to the control beam; the equivalent result for the EB-FRP system with and without end anchorage was 62% and 46% respectively.

Tan and Saha (2007) mechanically anchored pultruded FRP strips to an RC beam using MA-UFRP mechanical fasteners. They found the system to be less labour intensive, less time consuming and, in addition, no preparation was required of the concrete surface; these advantages led to a more economical solution than that of the conventional method of EB-FRP. A disadvantage of the mechanical fasteners is that they might lead to cracking in the concrete. When the upgraded system was placed under a cyclic load the stiffness of the strengthened beam degraded quickly. The authors suggested that this result was due to the degradation in the anchorage bond between the mechanical fasteners and the concrete as well as by the cyclic load.

Near surface mounted (NSM) FRP composite reinforcement technique. This is another method for rehabilitating RC structural members. CFRP, AFRP and GFRP composites

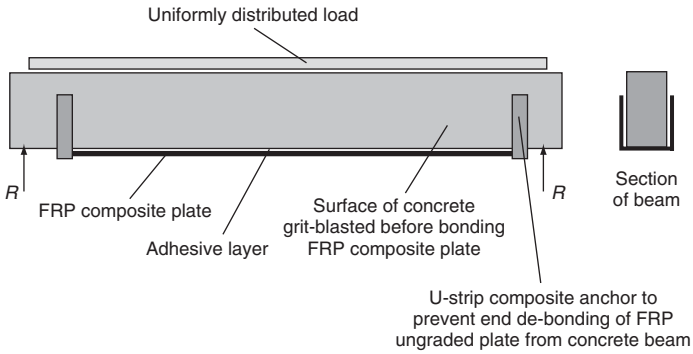

Figure 6 Illustrates the upgrading of an RC beam utilising FRP composites

can be utilised and generally the cross-section of the member is either circular or rectangular. Grooves are cut into the surface of the member, generally into the soffit of the concrete beam, but if the cover to the steel rebars is insufficient for this operation to be performed the grooves may be cut into the vertical side of the beam as near to the bottom of the section as is practical. The NSM FRP reinforcement is embedded and bonded into this groove with an appropriate binder (usually high-viscosity epoxy or cement paste). A review of the NSM reinforcement technique has been given by De Lorenzis and Teng (2007). **Figure 7** shows the position of the NSM bars in an RC structural member.

De Lorenzis *et al.* (2002), El-Hacha and Rizkalla (2004) have shown through experimental test results that NSM reinforcement can significantly increase the flexural capacity of RC elements. Bond may be the limiting factor on the efficiency of this technique as it is with externally bonded laminates. El-Hacha and Rizkalla (2004) compared the strengthening of reinforced-concrete T-beams with identical NSM strips and externally bonded plates. They obtained a strength increase of 4.8 times higher in the first

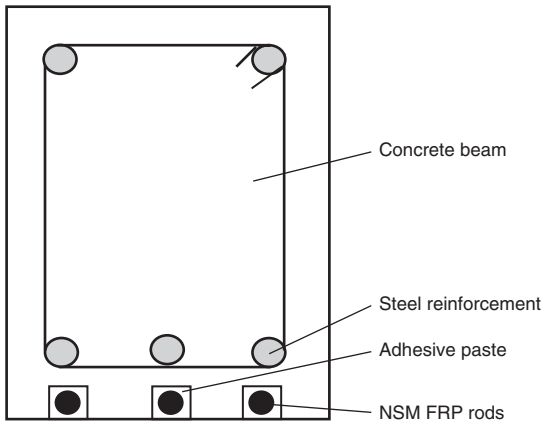

Figure 7 The position of the NSM bars in an RC structural member

case, due to an early de-bonding failure of the external FRP compared to a tensile rupture of the NSM strips.

NSM FRP reinforcement has also been used to enhance the shear capacity of RC beams. In this case, the bars are embedded in grooves cut into the sides of the member at the desired angle to the axis. Utilising NSM round bars, De Lorenzis and Nanni (2001) have shown experimentally that an increase in capacity as high as 106% can be achieved, thus when stirrups are used a significant increase can be obtained; however, in these particular tests, de-bonding of one or more of the FRP bars resulted and this was associated with concrete diagonal tension. It was stated that by providing a larger bond length this failure mechanism could have been prevented.

Stressed FRP plates

When externally bonded CFRP composite reinforcement is used to strengthen an RC beam, only about 50% of the strength of the composite can be exploited due to the problems of de-bonding (i.e. the concrete cover separation mode and the plate-end interfacial de-bonding mode) these result in the loss of composite action before the classical modes develop. These failures are undesirable as they are brittle in nature and occur at lower load levels than the classical failures. Furthermore, the intermediate flexural crack-induced interfacial de-bonding and the intermediate flexural shear crack-induced interfacial de-bonding are two further failure modes. However, the plate-end de-bonding failure modes can be prevented by utilising anchoring devices, or adopting suitable factors of safety in the initial design. A more efficient system, as far as the FRP material is concerned, would be to prestress it, thus providing the benefits of excellent plate durability, structural improvement and a more efficient use of the material capability. By prestressing, the externally bonded CFRP plate is used as active reinforcement to exploit more of the high tensile capacity of the fibres. The RC beam in the tensile zone is compressed and the stress in the internal steel reinforcement is accordingly reduced. Other advantages of this technique are a decrease in crack size, crack distances and a higher stiffness of the element. Crack control is important since it limits the deflection of the beam but also improves the durability of the structure. Moreover, not only are advantages realised at the serviceability state level but also at ultimate state as it is possible to reach higher strains in the composite resulting in higher ultimate strengths. The application of composite prestressing on the construction site is complex and time consuming and as a consequence the technique is currently used only in very particular circumstances. However, further research in this field will enable more confidence to be gained and more prestressing systems for field applications to be developed. Researchers in this field have included Quantrill and Hollaway (1998), Garden and Mays (1999), Wight *et al*. (2001), Wight and Erki (2003), El-Hacha *et al*. (2003), Piyong *et al*. (2003) and França and Costa (2007).

Stressed SFRP sheet systems

Prentice and Wight (2006) undertook an experimental programme to investigate the possibility of RC beams strengthened with externally bonded non-prestressed and prestressed steel-reinforced polymer (SRP) sheets; these have been compared with similarly bonded CFRP sheets. It was shown that prestressed sheets are more able to delay the deformation of cracks, reduce deflections and increase strength during yielding and at ultimate conditions. Initial tests on strengthened RC beams with SRP wraps undertaken by Carriere *et al*. (2007) have indicated that SRP may have a much higher lateral shear strength and a greater toughness than CFRP composites, thus making them more suitable than CFRP composites for strengthening concrete components subjected to blast loading.

Seismic retrofit of RC columns

The attractiveness of FRP composites due to their lightweight, tailorability characteristics and the significant advantages for use in seismic regions provides immense advantages in the development of structural components for bridges and buildings. The retrofit of RC structures improves the strength of those members which are vulnerable from seismic attack; it is possible to achieve a global performance characteristic by the failure of the more ductile and energy-dissipating components.

The seismic retrofit of RC columns tends to change the column failure mode from shear to flexural failure, or to transfer the failure criteria from column to joint and/or from joint to beam failure, depending upon the strengthening parameters. This technique is used in existing reinforced-concrete columns where insufficient transverse reinforcement and/or seismic detailing is provided; three different types of failure mode can be observed under seismic input. These are:

1. *Column shear failure mode*: this mode of failure is the most critical one. The modern seismic column designs contain detailed transverse or shear reinforcement, but the shear strength of existing substandard columns can be enhanced by providing external shear reinforcement or by strengthening the column through composite fibres in the hoop direction. The shear capacity of columns requires to be checked at the column ends and at potential plastic hinge regions where the concrete shear capacity can degrade with increasing ductility demands. It also requires to be checked in the column centre portion between flexural plastic and/or existing built-in column hinges.
2. *Confinement failure at the flexural plastic hinge*: subsequent to flexural cracking, the cover concrete will

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crush and spall, this is followed by buckling of the longitudinal steel reinforcement, or a compression failure of the concrete which in turn will initiate plastic hinge deterioration. These series of failures are less destructive because of their large, inelastic flexural deformations and therefore they are more desirable than the brittle column shear failures of the entire column as described in item 1. This ductile flexural plastic hinge at the ends of the column can be achieved through confinement by an increase in FRP hoop reinforcement.

3 *Confinement of lower ends of columns*: some bridge columns have lap splices in the column reinforcement; these are starter bars used for ease of construction and are located at the lower column end to form the connection between the footings and the columns. This is a potential plastic hinge region and it is advantageous to provide confinement by external jacketing or continuous fibre winding in this area.

None of these failure modes and associated column retrofits can be viewed separately, since retrofitting for one deficiency may shift the seismic problem to another location and a different failure mode without necessarily improving the overall deformation capacity.

The confinement of RC columns can be undertaken by fabricating FRP composites using techniques such as the wet lay-up polymer composites, the semi-automated hot-melt factory-made pre-impregnated fibre or the automated filament winding. Seible *et al.* (1997) and fib Task Group 9.3 (2001), have discussed the prefabricated (pre-cured) elements in the form of shells or jackets which are bonded to the concrete and to each other to provide confinement. The wet lay-up and the prefabricated systems are generally placed with the principal fibre direction perpendicular to the axis of the member. The concrete column takes essentially the axial load provided the ratio of the circumferential to axial fibres of the composite is large. This allows the tensile strength in the circumferential direction to be virtually independent of the axial stress value. A review of the effectiveness of FRP composites for confining RC columns has been given in De Lorenzis and Tepfers (2003).

Upgrading of metallic structural members

As a result of research which commenced at the latter part of the twentieth century, metallic structures are now being upgraded with CFRP composite material (Mertz and Gillespie, 1996; Mosallam, 1998; Luke, 2001; Tavakkolizadeh and Saadatmanesh, 2003; Cadei *et al.*, 2004; Luke and Canning, 2004, 2005; Hollaway *et al.*, 2006; Photiou *et al.*, 2006; Zhang *et al.*, 2006).

The FRP composite plate material used for the bonding operation is either the ultra-high modulus (European definition) or the high modulus (European definition) CFRP, AFRP (Kevlar 49) or possibly GFRP composites and these will be fabricated by one of four methods, namely:

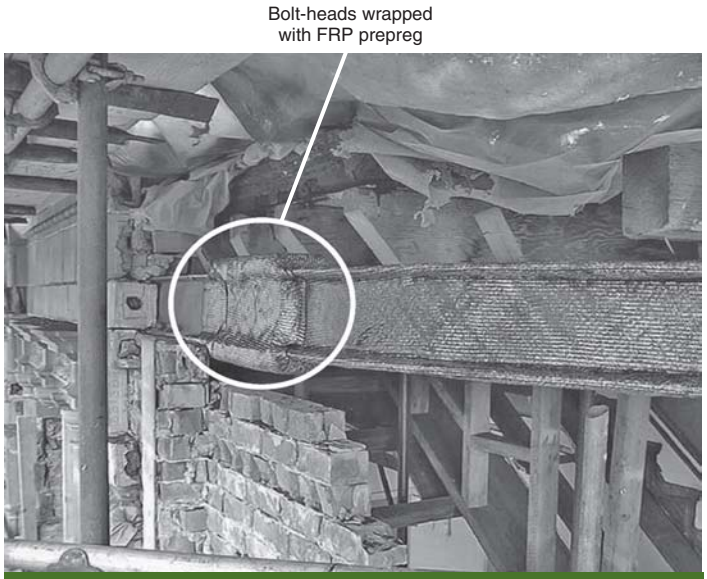

Figure 8 The upgrading of a curved steel structural beam by the hot-melt carbon fibre–epoxy composite prepreg, note the wrapping of bolted joint (courtesy of TaylorWoodrow, London and ACG Derbyshire, UK)

1 The pultrusion technique, in which the factory made rigid precast FRP plate is bonded onto the degraded member with cold-cure adhesive.
2 The factory made rigid fully cured FRP prepreg plate bonded to the degraded member with cold-cure adhesive.
3 The hot-melt FRP prepreg/adhesive film placed onto the structural member and both components compacted and cured under vacuum at an elevated temperature.
4 Vacuum infusion (the resin infusion under flexible tooling (RIFT) process).

Figure 8 shows the upgrading of a curved steel structural beam by the hot-melt carbon fibre–epoxy composite prepreg.

It should be mentioned that the ultra-high-modulus carbon fibre composite has a low strain to failure of the order of 0.4% strain, a modulus of elasticity of the composite of about 34 GPa and so the system will fail with a small inelastic characteristic. The high-modulus CFRP composites have a value of ultimate strain of the order of 1.6% strain for modulus of elasticity of 28 GPa. This implies that the material is ductile and is unlikely to fail in a rehabilitation situation by ultimate strain but by some other criteria (Photiou *et al.*, 2006).

FRP strengthening has been applied to struts and columns carrying compression. Moy (2004) has reported on the strengthening of cast iron cruciform-section struts

in a ventilation shaft on the London Underground using ultra-high-modulus CFRP composites, where the CFRP composite was designed to increase the capacity of the tensile zone of the strut during buckling. Shaat and Fam (2004) have discussed the use of FRP sheets to increase the local buckling strength of hollow square section columns.

Further information on the technique, analysis and design of the rehabilitation of CFRP composites to metallic structures may be obtained from Hill *et al.* (1999), *ISIS Design Manual* (2001), Liu *et al.* (2001), Moy (2001), Leonard (2002), CIRIA RP645 (2004), Photiou *et al.* (2004a, 2004b), Schnerch and Rizkalla (2004), Dawood and Rizkalla (2006).

Long-term mechanical properties of unstressed and prestressed FRP composites for strengthening of RC and PC structural beams

Fatigue loading behaviour of RC beams strengthened with unstressed FRP composites

There have been only a few studies published on the fatigue loading behaviour of beams:

- **a** strengthening with CFRP composites (Karbhari *et al.*, 2000; Hefferman and Erki, 2004);
- **b** on their response to a combination of fatigue loadings and low temperatures (Lopez *et al.*, 2001).

Furthermore, there are virtually no data dealing with the fatigue response of beams subjected to different fatigue load levels and number of cycles. Gheorghiu *et al.* (2004a,b) reported on the effect of severe fatigue loading on the flexural behaviour of such systems. During their fatigue testing the beams showed a consistent response independently of the number of fatigue cycles. The specimens maintained most of their initial stiffness until the end of the test. Initially the beams showed an important deflection increase up to 10^5 cycles. Then the response stabilised with deflections being asympototic to the maximum values corresponding to the load amplitudes; the beams were then loaded to rupture. The authors showed that the number of fatigue cycles had no significant effect on the load–deflection response of the beams. The total deflection, including the residual deflection due to cycling, is not significantly different from that of the control beams. Furthermore, the ultimate load reached by the beams does not appear to be influenced by fatigue cycling.

Fatigue loading behaviour of RC beams strengthened with prestressed FRP composites

Ma *et al.* (2007) have investigated the fatigue behaviour of RC beams strengthened with prestressed CFRP composites and have presented a design proposal for fatigue life based on the local stress–strain relationship of steel reinforcement in the beams. The conclusions from these tests were that the unstrengthened beam will have a high steel strain range and will lead to a shorter fatigue life, whereas the unstressed bonded plate to the concrete could prolong the fatigue life of the beam as the composite plate will decrease the steel stress range. The fatigue life will be further extended by the prestressed FRP composite plate as the steel stress range is further decreased and the prestressed plate will restrict the openings of the cracks. The proposed design was based on the local stress–strain relationships of the steel reinforcement in the beams and this gave a practical way of providing a precise prediction for the fatigue lives of RC beams.

Miller *et al.* (2007) undertook an experimental testing programme to repair PC bridge girders with FRP composites to evaluate the effectiveness of externally bonded CFRP sheets as a repair system for impact-damaged PC bridge girders. The programme consisted of testing three PC beams, one of 16.4 m and two of 5.4 m long; the former beam was repaired for impact damage at mid-span while the short beams were repaired for shear to simulate possible impact damage at or near the supports; this latter repair was based upon ACI 440.2R-02 (2002). The first damaged beam was repaired and tested under 2×10^6 cycles of fatigue loading. The authors commented that no stiffness degradation in the girder was observed; there was, however, some fatigue creep of concrete which caused a small displacement degradation totalling 2.8 mm. One of the two shorter beams was tested as a control specimen in which shear crack widths, displacement, compression strain and ultimate load capacity were measured; it failed in shear with diagonal cracks forming between the support and extending to the loading plate. The second short beam was damaged and repaired using both longitudinal CFRP sheets and 45° diagonal U-raps. The authors state that the ultimate capacity of the repaired beam exceeded that of the control beam; the failure mode was a combination of shear failure in the transfer region and plate-end de-bonding of the CFRP at the end of the girder away from the shear repair zone.

Loo *et al.* (2007) investigated a series of flexural PC beams to ascertain their fatigue capacity; these specimens consisted of three corroded, three corroded and then repaired with CFRP composites, and three control beams. The corroded beams became susceptible to fatigue failure and there was a 10% steel mass loss due to this environment with an increase in deflection of approximately 90% over that of the control beams at the same load cycle. The corroded beams were repaired to their original strength and stiffness. The control beams were nearing collapse after 1.2×10^6 cycles with wide growing cracks and a marked increase in deflection. At the same load the cracks

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in the repaired beam were fine cracks and there was no obvious strain distress in the specimen. It was concluded that the repair utilising CFRP composites is able to restore the fatigue life of a severely corroded beam up to and beyond that of an RC beam in its original uncorroded condition.

Fatigue loading behaviour of RC beams strengthened in flexure with NSM CFRP bars and strips

Yost *et al.* (2007) undertook experimental fatigue tests on RC beams strengthened in flexure with NSM CFRP bars and strips. The objectives of the tests were to observe how fatigue loading affected failure limit state, yield and ultimate strength, and flexural stiffness; the service level fatigue loading was 2×10^6 cycles on simply supported beams. Statically loaded beams were also tested for comparison. The authors report that at the end of the fatigue testing the beams were loaded to failure; all beams failed by steel yielding followed by concrete crushing. There was no evidence of bond deterioration as a result of fatigue loading or force transfer. Thus composite action between the NSM CFRP composite and concrete appeared to be unaffected under this loading and fatigue had no effect on the static failure limit state. There was an 8% increase in the yield load for the fatigue loaded bar and strip composite strengthened beams when compared to the static tested beams. One interesting result reported by the authors is that the ultimate capacity of the bar-strengthened fatigue samples was nearly identical to the static beams, whereas, the equivalent result for the strip-strengthened fatigue samples was about 5% less than the static beam. The authors suggested that fatigue loading resulted in a slight increase in yield load but had no affect on the section's ultimate static strength.

Cost associated with the use of advanced polymer composites for FRP strengthening of concrete bridge structures

Cost is arguably one of the most influential factors when assessing the merits of the utilisation of FRP composites in construction. However, detailed costing is difficult to provide impartially and can become quickly out of date in a changeable sector where new technology is continually emerging. Eyre *et al.* (2007) have discussed their work in developing a design process protocol software for the selection, analysis and cost profiling of suitable strengthening systems for the rehabilitation and retrofitting of structurally defective and/or deteriorating concrete bridge structures with FRP composites. The internal report of the University of Bath and University of Missouri-Rolla, entitled: 'FRP Strengthening System – Selection Toolkit', discusses the software that has been developed to give a rule-based, logic-programmed, expert-system. The main topics within the report are concerned with:

- deterioration and degradation processes;
- strengthening option selection and elimination;
- structural recalculation;
- FRP material and system technique choice;
- structural design;
- unit cost estimation.

The project was founded through the combination of the material, the system and the installation procedure selection, the inspection and the monitoring regimes. It was realised that accurate economic appraisal of solutions is vital to the validation of a preliminary strengthening scheme and therefore unit cost estimation is the ultimate objective of the software. The software facilitates extraction of a total repair cost per unit area and the final cost, generated through the developed software, provides typical material supply, labour and inclusive construction costs. The unit costs for these latter components are adjusted by a corresponding 'factor ratio' which is equated to the FRP material used. The effectiveness of the scheme is then defined as the percentage increase in analytical ultimate capacity after strengthening over the total cost per area (square foot) required to strengthen the structure, including company profits; the effectiveness is developed in De Lorenzis and Nanni (2005) and is shown in:

$$CE = (\%\ \text{Capacity increase})/(\text{cost per square foot})$$

Internal reinforcement to concrete structural members

The rebars used for reinforcing concrete are fabricated by the pultrusion method (Nanni, 1993; ACI 440R-96, 1996; Pilakoutas, 2000; Bank, 2006). The rebars are made from carbon, aramid and glass fibres using epoxy or vinylester polymers. As the surfaces of the composites manufactured by the pultrusion technique are smooth it is necessary to post-treat them in order to develop good bond characteristics between the concrete and the rebar. Several techniques are used to improve the bond; these include:

1. Applying a peel-ply to the surface of the pultruded bar during the manufacturing process. The peel ply is removed before encasing the bar with concrete, thus leaving a rough surface on the pultruded rod.
2. Over-winding the pultruded bar with additional fibres.
3. Bonding a layer of sand with epoxy adhesive to the surface of the pultruded rod; this is a secondary operation at the end of the pultrusion line.

Features and benefits of using FRP rebars include:

- Non-corrosive – will not corrode when exposed to a wide variety of corrosive elements including chloride ions or not susceptible to carbonation-initiated corrosion in a concrete environment.
- Non-conductive – provides good electrical and thermal insulation.
- Fatigue resistance – performs well in cyclic loading situations.
- Impact resistance – resists sudden and severe point loading.
- Magnetic transparency – not affected by electromagnetic fields.

FRP rebars manufactured from a thermosetting resin (i.e. vinlyester, epoxies) once polymerised are unable to be reshaped and therefore cannot be bent on site. If bends are required, for instance anchorages or stirrups, they must be produced by the FRP rebar manufacturer as a special item, but their strength at the bend will be considerably reduced. Thermoplastic polymers have been used as spliced bends; this material can be bent on site but the system is still in its development stage.

Although carbon and aramid fibre composites are higher in cost than that of the glass composite, they are inert to alkaline environment degradation and can be used in the most extreme cases. The chapter on Advanced Polymer Composites gives a discussion on the use of GFRP composite rebars in an alkaline environment.

Post-tension tendons

FRP composite tendons, with their non-corroding characteristics, have been proposed as an alternative to steel tendons in post-tensioning applications. CFRP composite tendons are characterised by their high strength-to-weight ratio, excellent fatigue resistance and low relaxation losses. A number of studies has been undertaken into the rehabilitation for prestressed concrete and cable stay bridges utilising prestressing bars and tendons made from FRP composites (Arockiasamy *et al.*, 1996; Fam *et al.*, 1997; Grace, 1999, 2000; Saadatmanesh and Tannou, 1999; Balázs *et al.*, 2000; Lu *et al.*, 2000; Burke and Dolan, 2001). Nordin (2004) has reported on a number of existing concrete bridge girders which have been rehabilitated and strengthened with external FRP tendons. In addition, Shehata *et al.* (1997) and Rizkalla *et al.* (1998) have undertaken experimental studies on Canadian precast concrete bridges that have been upgraded using prestressed FRP composite tendons; they have also illustrated the advantages of this prestressing system. The benefits of using FRP reinforcement compared with steel are its non-corrosive, non-magnetic, high strength and lightweight properties. However, many designers have been deterred from utilising FRP as tendons more widely because of their initial high cost and the increase in quality control required during their manufacture. Furthermore, despite their high tensile strength in the direction of the fibres, FRP composites are weak in the transverse direction; they are, therefore, highly vulnerable to premature failure at the anchor zone if any notching in the tendon occurs during the post-tensioning process or during service. This makes the conventional anchors used for steel tendons not suitable for FRP composites. Various anchor systems for FRP have been developed over the past two decades and generally the manufacturer recommends the type of anchor which should be used and invariably they are classified according to their gripping mechanism.

Several investigations have been conducted on various FRP tendon–anchor systems to evaluate their efficiency under static loading (Walton and Yeung, 1986; Nanni *et al.*, 1996; Al-Mayah *et al.*, 2001; Taha and Shrive, 2003a). In most cases, the anchor rather than the tendon itself controlled the ultimate capacity of the tendon with varying degrees of success in maintaining the applied load. However, it seems unlikely that, in the immediate future, FRP tendons will gain acceptance in construction without an initial economic incentive to use them; design guidelines for structural concrete reinforced with FRP composites. A review of the commercially available anchors can be found in Erki and Rizkalla (1993), Nanni *et al.* (1996), ISIS Canada (2001), Taha and Shrive (2003b) and ACI 440.4R-04 (2004).

The properties of FRP composite and steel tendons for designers have been given in Schupack (2001) and in ACI 440.4R-04 (2004). The properties that most affect the performance of prestressed tendons are strength, strain, creep and fatigue. Furthermore, the strength of FRP reduces as its size is increased due to shear lag effects as the surface bond stresses are transferred to the core of the tendon through matrix shear stresses. Excessive shear stresses in the matrix can initiate progressive failure of the tendons due to the relatively low shear strength of the polymer (GangoRao and Faza, 1992; Nikolic-Brzev and Pantazopoulou, 1995; ACI 440.4R-04, 2004). As FRP composite tendons are a polymer-based material with unidirectional fibre arrays they exhibit a linear elastic response up to failure and this will generally be observed to occur through fibre failure. This characteristic of the FRP composite will reduce the deformability of a concrete member prestressed with FRP composite tendons.

The data on the fatigue performance of FRP composite tendon–anchor systems are sparse, due mainly to the lack of an efficient anchor system. Uomoto *et al.* (1995) investigated the tensile strength and fatigue life of aramid, glass and carbon FRP composite tendons using split chuck anchors. Five load ranges, varying between 7% and 72% of the tendon strength, were considered under frequency loading varying between 1 and 10 Hz. The superiority of the fatigue strength of the carbon composite tendons over that of the aramid and glass composite tendons was

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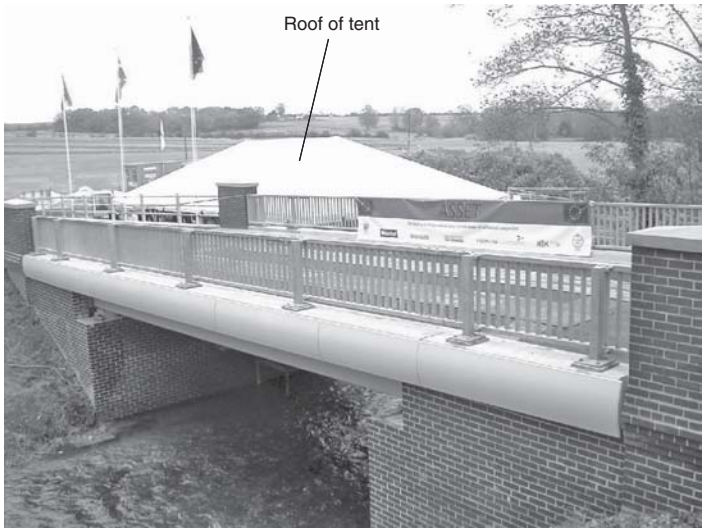
reported. The authors stated that the fatigue strength of the tendons was limited by creep failure at higher applied stress level. Saadatmanesh and Tannous (1999) reported on fatigue tests on Leadline and CFRP composites in tension–tension mode. Three different stress ranges and seven minimum stress levels were considered. It was stated that limited changes in the modulus of elasticity and in the Poisson's ratio had occurred when the tendons were subjected to a stress range of 5% of the ultimate strength of the tendon. However, when the stress range increased to 10% of the ultimate capacity of the tendon, an increase in the modulus of elasticity and Poisson's ratio of the tendon material was observed, indicating brittleness with increasing fatigue cycles. Dolan (1993) also noticed in his tests that with an increase in the number of cycles the carbon and aramid composite tendons also gained in stiffness, whereas the glass-fibre composite tendons lost stiffness. The researcher attributed the change in stiffness to the straightening of the fibres during the fatigue cycling. In addition, the FRP composite tendons have a good chemical stability in hostile environments and CFRP composite tendons have superior durability performance in moisture, alkaline and acidic environments compared with steel tendons. Furthermore, GFRP composites under a sustained tensile stress will stress corrode, therefore care must be taken not to stress GFRP composite tendons to a value greater than 25% of their ultimate strength. This phenomenon is referred to as creep-rupture. CFRP composite tendons have the lowest creep-rupture of the GFRP, AFRP, CFRP composite tendons, with a value of about 95% of their ultimate strength. The draping of the tendons that takes place in a prestressed girder also places a limit on the FRP tendon stress.

FRP bridge deck replacement

The FRP bridge deck is a component part of the bridge, and can be removed without causing damage to the primary component: the main structural beams (the superstructure). The superstructure can be manufactured from the more conventional civil engineering material or from fabricated FRP composites. The high-strength to low-weight ratio of the latter materials enables the bridge deck to carry the currently designed traffic loads with little or no upgrading of the superstructure. The dead load of the FRP bridge deck would be about 20% of the weight of an equivalent size of a concrete deck and can be erected within two days; moreover, the service life of the deck can be two to three times greater than that of the concrete decks. The FRP bridge decks, which contribute to the load-carrying capacity of the bridge, are suitable for the smaller span bridges up to about 20 m; the span of current bridges with FRP decks is generally about 10–12 m. If FRP bridge decks are to compete with concrete decks, the former decks must contribute to the load-carrying capacity of the top chords of the main girders; the concrete decks are generally built with complete composite action with the main girders. This construction increases the stiffness of the girder; this is achieved by shear studs or stirrup connectors. Therefore, FRP bridge decks must be constructed compositely with the main girders and in this case the joining would generally be undertaken by adhesive bonding. As a general rule the contribution of the FRP deck to the load-carrying capacity of the top deck chord can only be achieved with spans less than 20 m. Currently, there is some disagreement as to whether a composite action between the deck and the girders should be pursued or if a simple non-composite design should be used for bridges with FRP decks (Zhang and Cai, 2007).

- The first bridge deck to be erected in Europe was the bridge over the River Cole at West Mill, in Oxfordshire, UK in 2002. The span of the bridge is 10 m with a width of 6.8 m; the bridge carries two lanes of traffic and footpaths. This example is a complete replacement of an existing bridge deck and superstructure. The old bridge was demolished and the upgraded bridge now comprises reinforced-concrete spread footings and abutments, with brickwork facing, supporting four longitudinal pultruded GFRP composite box beams. The box beams have unidirectional CFRP composite flanges bonded to the GFRP beams to provide the required global flexural rigidity. The deck consists of a series of adhesively bonded advanced composite ASSET profiles (Canning *et al.*, 2002), spanning transversely, onto which the polymer concrete surfacing and epoxy wearing course for the carriageway are located. **Figure 9** shows the ASSET FRP bridge deck.



Span of bridge 10 m and width of bridge 6.8 m. Four main beams each manufactured from four GFRP pultruded square box sections of 225 mm sides and on each top and bottom face of the beams UD. CFRP composite plates are bonded. The bridge was tested with a 28 t lorry traversing the bridge before it was opened.

Figure 9 Asset FRP bridge over the River Cole at West Mill, Oxford UK (courtesy of Mouchel, West Byfleet, England)

- The first vehicle-carrying FRP bridge deck in the UK to span over a railway, replaces the existing over-line bridge at Standen Hey, near Clitheroe, Lancashire; it has a span of 10 m, weighs 20 t and was completed in March 2008. This is the first of Network Rail's six trial sites in the country. The consulting firm, Tony Gee and Partners, was responsible for the design of the bridge deck which comprises three layers of ASSET panel deck units which are made from E-glass fibres in the form of biaxial mats within a UV-resistant resin matrix.
- Composite advantage (CA) built (April 2008) a new 'drop-in-place' GFRP composite prefabricated integral beams and deck bridge superstructure, 22 ft long by 62 ft wide (6.77 m × 19.0 m) in Hamilton County, Ohio, USA. No heavy lifting equipment was required and it took one day to install (Composite Advantage Newsletter, 2008).
- The UK Highways Agency has recently completed a new single-carriageway road bridge over the M6 motorway. The superstructure comprises a novel prefabricated FRP deck spanning transversely over, and adhesively bonded to, two longitudinal steel plate girders. The FRP bridge deck was designed by Mouchel Group, Manchester, UK and provides general vehicular access to an equestrian centre; it was designed for unrestricted traffic loading (Canning, 2008).

Elastomeric bearings

Elastomeric bearing are designed to provide a flexible link between the two structural components which are to be isolated. To function adequately the elastomer must maintain a low shear modulus. The bearings to be discussed are divided into three groups; these are:

a bridge bearings;
b the rubber-based isolation system;
c anti-vibration and structural isolation systems against ground-borne vibrations.

Bridge bearings

Bridge bearings are used to accommodate and avoid damage from: (i) vehicle movements on bridges; (ii) thermal expansion of bridges; and (iii) loading to piers, thus reducing reaction forces and rotational movement to within safe limits. Bridge bearing movements are accommodated by the basic mechanisms of internal deformation

- Elastomeric materials – depending upon the specifications requested, neoprene polymer, natural rubber or styrene butadiene rubber (SBR) are ideal engineering materials to use in the form of plain pad and strip bearings or in lamination form with steel plates. The elastomeric bearings are required to be flexible in shear, and stiff in compression to allow the deck to translate and rotate but also to resist loads in the longitudinal, transverse and vertical directions. The stability of the bridge bearings must be taken into account in the design and they must be able to absorb and isolate energies from impacts and vibrations. International standards for elastomeric bridge bearings are given in BS 5400, Part 9, BS EN 1337-3 – European, (AASHTO), Department of Transport – BD20/92. **Figure 10(a)** illustrates a typical elastomeric bridge bearing. A diagrammatic sketch of a plane sliding bearing is shown in **Figure 10(b)**. The material used with this system is the low friction polymer polytetrafluroethylene (PTFE), which slides against a metal plate. This bearing does not resist rotational movements in the longitudinal or transverse directions, only loads in the vertical direction. The longitudinal and transverse loads can be resisted by providing mechanical keys.

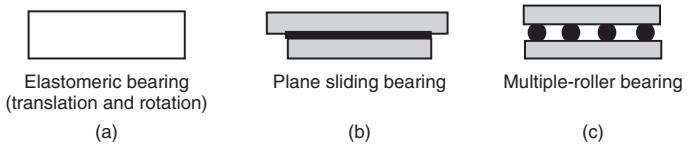


Figure 10 Illustrations of different types of bridge bearings

- Roller bearings – a typical multiple-roller bearing is shown in **Figure 10(c)**. Large longitudinal movements can be accommodated by these bearings, but vertical loads only can generally be resisted. The material of the rollers tends to be steel and therefore, this type of bearing is outside the scope of this book.

The rubber-based isolation system (seismic isolation)

The seismic isolation system is to introduce flexibility at the base of a building structure in the horizontal plane, while at the same time introducing damping elements to restrict the amplitude of the motion caused by an earthquake. The technique can significantly reduce both floor accelerations and inter-storey drift and provide a viable economic solution to reduce non-structural earthquake damage. There are three basic elements in a seismic isolation system. These are:

- A flexible mounting so that the period of vibration of the total system is lengthened sufficiently to reduce the force response.
- A damper or energy dissipater so that the relative deflections between building and ground can be controlled to a practical design level. Elastomers with high damping characteristics could be used for this element.
- A means of providing rigidity under low service load levels such as wind and minor earthquakes.

The rubber-based isolation system consists of either high damping steel–rubber or lead–rubber laminations of thicknesses between 160 mm and 200 mm. The former type of base-isolation technology is used in China but the latter type is the most popular device and is used mainly in the USA, Japan, New Zealand and Italy. The typical residential buildings using the lead–rubber type are reinforced concrete frame or wall construction of more than five storeys high. Other systems use elastomeric pads constructed of neoprene layers in series and are available with alternating raised diagonal ribs or square cell pattern. An alternative seismic isolation system is the passive energy

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dissipation technique. These systems do not use elastomeric materials and this technology will not be discussed here. Those buildings that do not meet the seismic code requirements or that were damaged by an earthquake would tend to be retrofitted using FRP composites.

Anti-vibration and structural isolation systems

There are a number of building constructions that require isolation from ground-borne vibrations in inner-city redevelopments due to low-frequency rumble from underground and surface trains where there is a risk of sound transmission from rolling stock. These vibrations not only threaten the foundation structure, they also pose a risk of disturbing occupants in adjacent buildings. In these cases the isolation of the structure can be effected by placing elastomeric bearings (such as polyurethane-bound rubber granulate, polyurethane mixed-cell structure foam, a medium-density closed-cell structural foam such as isolation sheets, etc.) under its foundations. The isolators are installed on top of the basement walls or columns.

Applications of this technology include:

- foundation isolation;
- column heads;
- pile cap;
- perimeter isolations;
- floating floor systems.

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Appendix: Design codes, codes and specifications for the design of FRP composites in structural engineering

In recent years a significant number of design codes and specifications have been published by technical organizations which provide guidance for design with FRP materials for civil engineering. The key publications are listed below.

British/Europe

Design Guidance for Strengthening Concrete Structures Using Fibre Composite Materials, 2000, TR55, 2nd edition, Camberley, UK: Concrete Society.

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Chapter 59

Recycling of FRP materials in construction

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While recycling technology is developed for FRP materials, the current practice is to send the majority of waste produced to landfill. Legislation will force industry to address other options through the waste hierarchy, putting more pressure on solving FRP waste management through recycling and reuse.

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Introduction

Fibre-reinforced polymers (FRPs) are increasingly being used in construction due to their light weight, ease of installation, low maintenance, tailor-made properties and corrosion resistance. The UK FRP industry produces 240 000 t of products a year with 11% of this being for the construction industry. Current and impending waste management legislation will put more pressure on the industry to address the options available for dealing with FRP waste. Such waste legislation focuses on dealing with waste through the waste hierarchy and will therefore put more pressure on solving FRP waste management through recycling and reuse (Halliwell, 2006).

At present the most common disposal method for UK FRP waste is landfill. To assist in the transition from disposal in landfill to recycling, the FRP industry needs to consider designing materials and components for easier deconstruction, reuse and recycling at the end of the product life. Successful FRP recycling requires incentives, infrastructure, recycling techniques and market commitment.

FRP waste management practice – waste hierarchy

Landfill and incineration have always been the simplest and preferred methods of disposal accounting for 98% of composites waste, while alternative routes such as reuse and mechanical recycling account for the remaining 2% (GPRMC press release, 2003).

New European waste directives on landfill and incineration will put mounting pressure on these traditional disposal routes with threats of bans on landfill of FRP waste by most EU Member States and energy limits on incineration. All waste legislation focuses on dealing with waste through the waste hierarchy (**Figure 1**) and will therefore put more pressure on solving FRP waste management through recycling and reuse. In order to comply with such legislation, manufacturers of FRP products will need to have products for which real recycling solutions exist.

Volumes of waste produced by the total FRP market were estimated to be 156 000 t in 2000, with around 70% of this coming from end-of-life waste and the remainder from production waste. It is thought that these figures could almost double to a total of 304 000 t in 2015, with over 80% of this arising from end-of-life waste (Harbers, 2002). Other figures of at least 500 000 t of FRP being scrapped in Europe have also been estimated. The amount of waste produced by FRP manufacturers is dependent on the type of material and the manufacturing process (SICOMP, 2002). Currently, the total amount of collected FRP waste in Europe is around 400 t per year, of which 300 t is used in bulk/sheet moulding compound (BMC/SMC) for non-visible automotive applications, and 100 tonnes is used as an additive in concrete to avoid crack formation during the drying process.

Legislation

The increasing volumes of all types of waste are leading to concern over methods of waste disposal and ways in which waste can be prevented. This has resulted in the creation of directives and regulations at European and UK government levels which relate to the efficient management of waste through implementation of the waste hierarchy.

EU waste management legislation

The EU waste management Directive provides the overarching legislative framework for the collection, transport, recovery and disposal of waste, and includes a common definition of waste (EU, 1975). The Directive requires all Member States to take the necessary measures to ensure that waste is recovered or disposed of without endangering human health or causing harm to the environment and includes permitting, registration and inspection requirements. The Directive also requires Member States to take

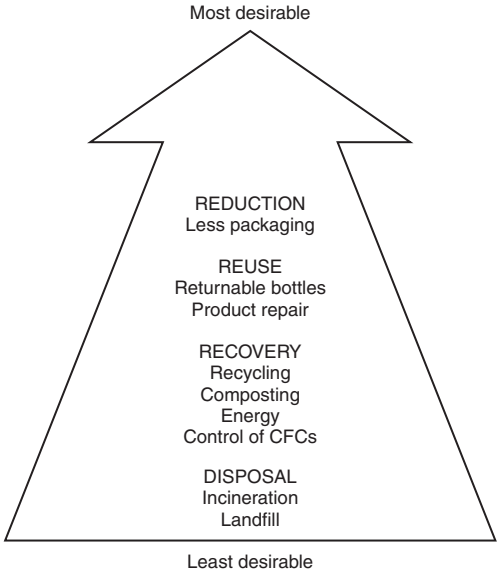

Figure 1 The waste hierarchy

appropriate measures to encourage, first, the prevention or reduction of waste production and its harmfulness, and, second, the recovery of waste by means of recycling, reuse or reclamation or any other process with a view to extracting secondary raw materials, or the use of waste as a source of energy. The Directive's overarching requirements are supplemented by other Directives for specific waste streams.

On 27 May 2003 the Commission adopted a Communication 'Towards a Thematic Strategy on the Prevention and Recycling of Waste'. In the document (EU, 2003), the Commission sets out a wide range of suggestions and ideas for the possible future development of policy on waste in the EC. The Communication aims at developing a comprehensive strategy which includes prevention targets and measures needed to achieve them. For waste recycling, this Communication investigates ways to promote recycling where potential exists for additional environmental benefits and analyses options to achieve recycling objectives in the most cost-effective way possible.

UK government policy – Waste Implementation Plan

The Waste Implementation Programme (WIP) responds to the package of strategic measures recommended by the Strategy Unit (SU) report 'Waste Not Want Not' published in November 2002 (DEFRA, 2002), and the Government's Official Response (HM Government, 2002). The remit of the SU was to consider action to be taken to help the UK to meet the legally binding targets under Article Five of the EU Landfill Directive.

Taking account of the derogations available to the UK, the targets are:

- By 2010 to reduce biodegradable municipal waste landfilled to 75% of that produced in 1995.
- By 2013 to reduce biodegradable municipal waste landfilled to 50% of that produced in 1995.
- By 2020 to reduce biodegradable municipal waste landfilled to 35% of that produced in 1995.

Landfill directive

The aim of the landfill directive is to prevent or reduce negative effects on the environment and human health from the landfilling of waste, during the whole life-cycle of a landfill (EU Landfill Directive, 1996). The directive states that the price charged for disposing of waste in landfill should cover the costs of setting up and operating the site as well as the estimated cost of closure and after-care of the site.

Landfill tax

The landfill tax credit scheme (LTCS) was introduced with the landfill tax in 1996 (LTCS, 1996) and applies to waste disposed of in licensed landfills. The LTCS was designed to help mitigate the effects of landfill upon local communities. The aim of landfill tax is to ensure that the price of landfill fully reflects the impact that the landfilling of waste has upon the environment. The money raised from the landfill tax is used to encourage the use of more sustainable waste management practices and technologies. The government has stated that the standard rate of tax will rise by £3 per tonne per annum to a rate of £35 per tonne in 2010.

Incineration of waste directive (Defra, 2008)

The aim of this directive is to prevent or limit the negative effect on the environment and resulting risks to human health, from the incineration and co-incineration of waste (incineration directive). Limits are set concerning levels of emissions to air, water and soil. Residues from the incineration process will be minimised in their amount and harmfulness and recycled where appropriate. At present, the apparent environmental benefits of energy recovery by burning plastics such as GRP in a waste incinerator are likely to be outweighed by significant drawbacks.

Proposed recommendations for construction and demolition waste

In 2001 the Office of the Deputy Prime Minister carried out a survey into the arising and use of construction and demolition (C&D) wastes (Halliwell, 2006). This followed an initial survey in conjunction with the Environment Agency in 1999/2000. Total construction and demolition waste for England and Wales was estimated at 93.91 million tonnes, up from an estimated 72.5 million tonnes in 1999. Forty-eight per cent was recycled and a further 48% was beneficially reused, mainly for layering or topping at landfill

sites and backfilling quarries. The remaining 4% was sent to landfill as waste.

The European Commission wishes to introduce a recommendation (a non-binding measure) for C&D of waste with the aim of improving the management of the C&D waste stream by following the waste hierarchy, giving preference to prevention over reuse, material recycling, energy extraction and lastly disposal. It will aim to reduce the impact of C&D waste on the environment while better utilising natural resources.

The recommendation will also encourage the substitution of hazardous substances in new buildings and make sure that waste from construction (bricks, glass, wood, etc.) is sorted at the point of generation. It would also include proposals for recycling targets set initially at 50% and an increase in landfill charges. It is thought that the European Parliament would prefer binding legislation rather than just a recommendation. So far progress has been very slow. If adopted, this recommendation/legislation will have a significant impact on the composites industry which supplies the construction sector.

Additional EU directives

EU directives such as End of Life Vehicles (ELV) and Waste Electrical and Electronic Equipment (WEEE) will put more pressure on solving FRP waste management through recycling and reuse. The ELV directive states that, by 2015, 85% of ELVs will have to be reused or recycled (excluding energy recovery), with only 10% incinerated with energy recovery, and only 5% going to landfill. While this new legislation does not impact on the construction industry, currently in negotiation is the proposed EU recommendation on Construction and Demolition Waste, which if adopted will have a significant effect. FRP suppliers could lose their market share to metal and other industries if they cannot ensure that their FRP components can be reused or recycled at the end of their life.

Life-cycle assessment and cost implications of recycling or disposal

Life-cycle assessment (LCA) is a method of assessing and quantifying the environmental impact of products taking into account the methods and materials used for manufacture. It also considers the impact on the environment during use of the product and its disposal, whether through recycling, incineration or dumping in a landfill site (De Vegt and Haije, 1997). Various life-cycle assessment tools are available:

- *Green Guide to Composites* (NetComposites, 2008): this guide has been created to allow the FRP industry to understand the environmental and social impacts of different materials and manufacturing processes.
- *SimaPro 4* (PRé Consultants BV, 2008a): SimaPro is a full-featured LCA software tool. It enables analysis of take-back and disassembly of products, as well as complex waste treatment and recycling scenarios.
- ECO-it: (PRé Consultants BV, 2008b): Eco-indicator tool for environmentally-friendly design. ECO-it allows you to describe a complex product and its life cycle.

Waste minimisation

The most cost-effective and environmentally beneficial option of waste management is not to produce the waste in the first place. By reviewing the manufacturing process it may be possible to identify a method which results in less production waste.

Waste takes a variety of forms such as offcuts, overspray trimmings, trimming dust, trimming from vacuum infusion, defective items and trials runs, plus obsolete moulds. Most automated processes are very efficient and there is little scope for improvement in terms of reducing generated waste, although waste may be generated at the beginning and end of production runs, or if components fail to meet accepted standards, e.g. through a faulty set-up.

Most of the waste produced is disposed of by landfill. Some manufacturers bear the cost of sending bulky waste by way of a skip container to landfill using a contractor, and there is considerable scope for reducing this burden (**Figure 2**). Contamination of FRP waste which could be recycled with other waste such as resin containers, release agent, rainwater, cleaning rags, etc. is an important issue.

Reuse of FRP materials

Reuse is high in the waste hierarchy, but may not provide the most practical option for many FRP applications. The way in which a material is used, its applications and how it is secured to other components must be considered with a view to deconstruction and reuse at the end of that application's life. The manufacturing process must be

Figure 2 Disposal of waste FRP

Figure 3 Reuse of GRP cabin

examined to identify any possible modifications to enable design for future reuse or recycling.

Issues associated with material reuse

Many items made from FRP are bespoke in nature, being especially designed for a particular application. This means that it is very unlikely that such products will be able to be reused for another, different, application. Some downgrading of product use may be possible (e.g. to agriculture). Some unusual or novel reuse of bespoke FRP features may occur. Reuse already takes place for items such as cabins and gatehouses in the construction sector (**Figure 3**), and pallets in the packaging sector.

Structural items such as sections or I-beams may be difficult to reuse since it is difficult to recalculate their load-carrying properties as recovered items, or reliably access any degradation or creep effect. This is quite different from the situation for recovered steel girders, large-section timbers and old bricks, for example, all of which are reclaimed and have a market value today. Without reference to the original manufacturer it will be difficult to derive the strength characteristics such as shear and bending of an FRP section with unknown matrix–fibre combination and make-up. FRP sections also tend to be produced to meet a particular set of circumstances and conditions so will often not be transferable to a different use. The designer of a building or structure will have a duty of care to make sure it is sound and a material cannot be reused if its strength properties are unknown or in doubt. This also applies to any fixings, bolt holes etc. for FRP claddings and mouldings. Similarly, care must be taken with reuse of cladding or roofing to ensure its fire-resisting properties are known.

Development of modular and prefabricated systems should allow reuse if this aspect is considered at the design stage, especially in respect of the type of site-applied sealing and gluing of joints.

Although FRP manufacturers have largely solved issues relating to UV stability and colour fading of their products, many fascia panels and mouldings may be surface degraded. This will affect the ability to reuse such components. FRP claddings and mouldings can be refurbished, by painting for example. FRP components are economical to produce, and this counts against reuse of existing items if they require labour-intensive inspection, cleaning, decontamination or repair.

There is presently little or no market for used FRP components. For reuse to be successful there must be an established supply chain.

Re-certification

Re-certification of composites products is unlikely to be cost-effective if testing is involved. For example, the cost of testing a trussed rafter is around £1500 for a product which costs around £30 to manufacture. Re-certification by the original manufacturer may not be in their financial interest.

Recovery of FRP waste

To make recycling procedures as easy and cost-effective as possible, FRP waste needs to be recovered in as clean and pure a condition as is feasible (Halliwell, 2006). This can be difficult to achieve since the materials are often used in association with other materials, often being joined together by adhesive or mechanical means. Materials need to be clearly labelled for rapid and accurate identification of component contents. Thus deconstruction of structures and components in order to recover FRP waste is a key step in the recycling process. After use, materials may be dirty and require careful cleaning, which is a costly, labour-intensive task. They may also be contaminated (e.g. with sewage or chemicals). The reclaimed materials will require storage and could be quite bulky.

Recycling technologies

There are essentially four classes of recycling technique which apply to FRP waste in general:

- Primary recycling – conversion of waste into material having properties equivalent to those of the original material.
- Secondary recycling – conversion of waste into material having properties inferior to those of the original material.
- Tertiary recycling – conversion of waste into chemicals and fuel.
- Quaternary recycling – conversion of waste into energy.

Most tried techniques for primary and secondary recycling essentially involve mixing of some waste material with virgin raw material, which is then processed as if it were all virgin material. Whether the proper designation

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for the procedure is primary or secondary recycling is a matter of how successful it is. Tertiary recycling refers to chemical decomposition of the polymer, i.e. depolymerisation, into useful chemical substances and fuel, while quaternary recycling is synonymous with incineration with utilisation of the energy released. The technique of incineration without energy utilisation is not really recycling at all, merely a means of reducing waste volume.

From the point of view of material utilisation it is generally preferable to succeed with the highest possible level of recycling, e.g. secondary rather than quaternary recycling. However, from economical and overall resource utilisation perspectives this need not necessarily be the case, since for example secondary recycling may require excessive amounts of energy and other resources (facilities, man power, chemical additives, etc.), while quaternary recycling is straightforward and does not require energy or specialised resources.

Primary and secondary recycling

The most attractive recycling option is to recycle waste into material with properties equivalent or at least comparable to those of the original material (as common with metals and glass) since it implies a possibility for an infinite number of recycling circuits. The division between primary and secondary is relative and a question of what extent of property degeneration that one is able to allow while still saying that the properties are equivalent. Common alternative names for primary and secondary recycling are material recovery or mechanical recycling.

Although it may be conceptually less straightforward to recycle thermoset FRP than thermoplastic, the common misconception that thermoset FRP cannot be recycled due to the presence of crosslinks is a myth. Proven technologies for material recovery are based on shredded then ground or milled FRP used as filler or possibly as replacement for some of the reinforcement in FRPs that generally end up having properties inferior to those in the original material.

To enable material recovery of thermoset FRP, the component to be recycled must first be reduced in size. With a large component the first step is generally to saw or break it into manageable pieces that can then be fed into a shredder to further reduce their size to the order of tens of centimetres. Prior to shredding, all metal inserts and fasteners must be removed. The shredded material is then ground to small pieces or milled to a powder; the final particle size may be anywhere between several millimetres to fractions of a millimetre. This regrind is separated by size using gradually courser meshes. In coarse regrind the reinforcement remains partially intact and thus the regrind may be used for reinforcing purposes, while the fine powder may only be used as a filler.

Ground FRP waste can be recycled as secondary raw materials for SMC and BMC, and also as filler or reinforcement. Only good-quality FRP waste with guaranteed formulation and no contamination is suitable for this, for example manufacturing waste. Using ground FRP recyclate in SMC and BMC does not have a detrimental effect on the mechanical properties of the product as long as the recyclate is used under certain ratios (no more than 20% for SMC and 30% for BMC). However, the low-value use of recyclate in this way means that it is difficult to recover the cost of the recycling process (Hayashi *et al.*, 1998).

There is concern that more complex FRP products such as sandwich panels may be more difficult to recycle with separation of the components necessary. The outside skins must be easily separated from the core to enable recycling. A study in Spain (Gaiker, 2009) has shown that sandwich panels produced using unsaturated polyester resin transfer moulded (RTM) skins and PET foamed cores can be recycled.

In the UK, the RRECOM project (RRECOM, 1994–1997) focused on strategies and technology for the reuse of suitably comminuted (ground) thermoset recyclate as a functional filler for polymers leading to products with added value (Bevis, 1996). An integrated continuous compounding process was developed which included comminution, drying and volatile extraction, combination of reinforcement with the matrix, optimum mixing, and die forming of the material. The recyclate was combined with the polymer matrix ensuring that effective dispersion and wet-out occurred. A range of polymer compositions was created containing fibre-reinforced polyester and phenolic recyclate derived from industrial scrap. The thermoset recyclate fillers were successfully incorporated into polypropylene and an enhancement in mechanical properties relative to the unfilled polymer was achieved. Reinforced phenolic recyclate was successfully incorporated into polyester resin and improvements in fire performance (reduction of smoke emission) relative to unfilled polyester resin were obtained. The study suggested that integrated compounding technology provides a cost-effective route for realising value from reinforcing thermoset recyclate fillers.

Conversion of waste into chemicals and fuel – tertiary recycling

Pyrolysis

Pyrolysis is the heating of waste in the absence of air (oxygen) and is used to separate composite material into its original constituents. The process breaks FRP down into gas, oil, fibre and a small amount of carbon. The oil and fibre obtained can both be reprocessed into FRP. Again, the operating temperature must be tailored to the type of FRP being processed in order to minimise thermal degradation of the fibres recovered.

Pyrolysis can be viewed as a means of recycling FRP waste and many trials have been successfully conducted. However, the European Commission does not consider pyrolysis to be a recycling process (Conroy *et al.*, 2004).

The evolving technology of pyrolysis is thought to offer a route for the recovery and reuse of both the hydrocarbon and inorganic content of FRP, producing recovered streams in the optimum condition for reuse. In particular, the process temperature and subsequent separation stage can be tailored to liberate the component materials whilst minimising thermal degradation. It is expected that the pyrolysis-based recycle route will be applicable to all FRP wastes. The materials recovered will be optimised and their recycling and reuse fully evaluated to achieve viable and sustainable recycling routes with the potential for commercial uptake. A cost-effective recycling process is an essential step towards sustainable manufacturing.

Supercritical water processing

Supercritical water processing can hydrolyse and decompose FRP effectively and cleanly without charring. The process involves heating waste composite in steam at 300 to 500°C resulting in the material being decomposed and partially hydrolysed to phthalic acid, styrene, glass fibre and oil. A supercritical water system for recycling FRP waste can be considered if its economics are reasonable. Such a system should also be able to take all types of FRP, including those with epoxy or phenolic resins for example (Bevis *et al.*, 1996).

Fluidised-bed processing

A number of studies have looked at the use of fluidised-bed thermal processing techniques for recycling FRP resulting in materials and energy recovery. A fluidised bed is a chamber containing sand which acts like a fluid when suspended in an air-stream. The chamber is heated to between 450 and 500°C, too low and the fibres will not be fully cleaned, any higher and the fibres suffer a reduction in strength. Chopped-up FRP material is placed in the bed. The resin is evaporated and the fibre is blown by the air-stream to a collection point for recovery (Global Composites, 2002). Gas enters a secondary combustion chamber for heat recovery leading to clean flue gas and recovered energy.

An economic analysis of the fluidised-bed recycling process showed that an operation recycling in excess of 10 000 t of glass-fibre composite material per year would be commercially viable (Bevis *et al.*, 1996). These are quantities that do not exist in the UK at present.

Further studies have been conducted by the University of Nottingham and Boeing regarding the recycling of carbon fibre FRP which is potentially a source of more valuable fibre for reuse (Carberry and George, 2007; Wong *et al.*, 2007). Prepreg offcuts currently provide the greatest source of carbon-fibre-reinforced polymer scrap. Carbon-fibre composites generally do not contain fillers, only fibres and resin. These composites usually require the fluidised bed to be operated at a higher temperature (450 to 550°C) as the epoxy resins they use are broken down at a higher temperature. Results have shown that the tensile strength of carbon fibres recovered is reduced by 20% at operating temperatures of 450°C. A carbon-fibre recycling plant has the potential to be viable at much lower annual throughputs than one for glass-fibre composites, and may show a more favourable prospect of being viable in the short term.

Incineration – quaternary recycling

When FRPs are combusted with energy recovery, the calorific value will depend on the inorganic content. Although the combustion of FRP waste is thought to be cleaner than that of coal, there is still a bulky ash residue. For example, in a typical GRP product 40% is glass, 30% inorganic filler and 30% resin. The glass and the filler do not burn, leaving 70% of the material as a residue after incineration. It has also been suggested that incinerating FRP material with energy recovery results in energy loss (in the region of −400 kJ/kg). Together, these mean that incinerating GRP is not a long-term solution for dealing with this waste stream. Natural-fibre-reinforced polymers are thought to be the best type of composites for incineration with energy recovery, as the natural fibres and fillers should burn easily, leaving no residue and resulting in no energy losses (Joshi *et al.*, 2004).

Co-incineration of GRP in cement kilns is thought to be a slightly better option as this offers combined material and energy recovery and therefore utilises the material more effectively. The resin is burnt for energy recovery, and the glass fibres and fillers become part of the cement, resulting in material recovery (Hayashi *et al.*, 2000). The cement kiln process means that no residue is left at the end. Companies wishing to use FRP waste in cement kilns should ensure that their emissions meet the necessary regulations (Defra Directive, 2007).

In addition to co-incineration in cement kilns, FRP waste can be co-combusted with coal in a fluidised bed. When limestone-filled FRPs are used in this way, energy is recovered from the polymers, and the limestone filler absorbs oxides of sulphur from the coal, making the process cleaner.

All incineration processes must comply with the European incineration directive (Defra, 2008) which aims to prevent or limit the negative effect on the environment and human health from the incineration and co-incineration of waste.

There are no specific limits or conditions for the incineration of FRP except for:

- FRPs containing more than 1% halogenated organic substances (for example, this may affect fibre-reinforced

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PVC, materials with >1% brominated flame retardants, sandwich structures with CFC foam).

- FRPs polluted during use (for example, GRP tanks used for storage of loaded gasoline or PAH-containing products).

These particular types of waste may be classed as hazardous and their incineration would have to comply with operation conditions. With regard to co-incinerating waste in cement kilns, it is thought that this is still possible after the introduction of the directive, but careful attention would have to be paid to Annex II of the directive.

Use of composite recyclate

Several potential uses for ground composite recyclate exist and much research is being carried out to develop high-value end-market applications.

Artificial woods have been manufactured using powder from pulverised FRP waste. The artificial woods were autoclaved from cementitious compositions with various other contents including carbon fibre. The material can be nailed and sawn like natural wood (Demura *et al.*, 1995).

Other research has investigated the use of scrap graphite/epoxy prepreg waste as a high-performance reinforcement for recycled HDPE (high-density polyethylene) plastics obtained from the municipal solid waste stream (George and Dillman, 2000). Flexural properties increased dramatically with increasing prepreg content and creep resistance was enhanced. It is thought that the product may find a niche in the plastics lumber market owing to the improved modulus, strength and creep resistance compared with current commercial recycled plastics lumber.

The UK highway engineering industry shows a willingness to adopt alternative sources of material for use in road construction and thereby promote sustainable development. As road construction consumes vast quantities of natural materials, it has the potential to provide a suitable end-use for composite waste. A joint RMCEF project (Woodward *et al.*, 2005) aimed to improve the recyclabilty of FRP building products by incorporating GRP waste into highway materials and conducting tests to ascertain whether they have a detrimental effect on the material performance. Preliminary findings showed that the addition of shredded GRP offcuts at 1% has had minimal effect on the performance properties of 20 mm dense bitumen macadam. The research is continuing using higher proportions of shredded composite material. Current research at The Building Research Establishment (BRE) involves collation of information on types, quantities and location of composite waste-streams and selection of samples for laboratory evaluation (BRE project). Testing will include determination of aggregate equivalent properties, modification of bitumen properties, optimisation of percentage addition to wearing course and base course bituminous mixtures, determination of mixture properties using gyratory compaction, and determination of stiffness, fatigue, creep and permanent deformation. Road trials will evaluate performance of base course and wearing course mixtures following guidelines laid down by the Highways Authority Product Approval Scheme (HAPAS).

In developing end uses for FRP recyclate, the products should utilise properties given by the recyclate, particularly where these give added value, such as:

- chemical and physical properties (e.g. phenolic FRPs for greater fire resistance);
- where recyclate can give special surface effects and designs;
- for noise absorption (wall panels or in roads);
- use of fluffy material for high thermal insulation or for non-woven materials;
- for viscosity modification of polymer mixes;
- use of coarse-grade material as a permeable flow layer in reinforcement for vacuum injection;
- low-cost core material (in boats or in wood substitute products);
- as reinforcement;
- milled fibres;
- asphalt reinforcement;
- for concrete repair (concrete is prone to cracking in the first two days of drying, use of glass avoids crack formation; there is a few thousand tonne market in Europe for this application);
- in white lines for road markings (abrasive resistance).

The development of new, high-grade markets is a high priority for the development of FRP recycling.

Composite recycling facilities

There are no FRP recycling facilities in the UK at present. Trials have been conducted in the UK to assess methods for material and energy recovery from FRP, including fluidised-bed processing, pyrolysis, grinding, and incineration/co-incineration. However, there are currently no facilities set up to conduct this on a large scale. There are processes in development that can use mixed polymer waste as raw material. These processes could also utilise composite waste, for example Plascrete which uses mixed waste polymer as an aggregate in a cementitious composite and can use glass-reinforced polymer. This is in trial production in the UK at present.

Facilities exist in other European countries, including Germany (ERCOM), France (MCR) and Norway (Miljotek), which grind FRP waste for reuse. However, these facilities are not greatly used and have little output. They also have difficulties in finding markets for their recyclate.

The feasibility of mechanical recycling has been proven on an industrial scale, but low utilisation levels mean that these ventures are currently uneconomical. Therefore, an

imbalance remains between the collectable waste and the potential end markets for recyclate. In order to meet these challenges, the key suppliers in the chain together with the European composite trade association, EuCIA, and associated bodies are introducing the 'European composite recycling concept' (ECRC website). This will help to fund the high investment needed to develop, validate and promote new recycling, reuse and end-of-life solutions for FRP waste.

A European composite waste management company is currently being set up via the Green FRP Recycling Label (ECRC website) which will coordinate the recycling of all European FRP waste. This pan-European approach developed by the EuCIA aims to address the waste management problems currently associated with composites by dealing with European waste in the most cost-effective way while respecting environmental and other obligations imposed by European, national and local legislation. It also aims to develop new, economically viable markets for FRP recyclate so that industry can meet the recycling targets imposed by specific European legislation.

European waste legislation will force a move away from traditional waste management through landfill and incineration, to reuse and recycling. This is likely to result in significant increases in the cost of waste management for the FRP industry. Under the Green FRP Recycling Label, licence fees are paid by the industry for the use of the label, which in turn finances the waste management services. Products with the Green FRP Recycling Label are guaranteed to possess a recycling concept according to the latest European waste directives. Recycling is free of charge for products that have the label, whereas a charge will be made for recycling products without the label. It has been suggested that the Green FRP Recycling Label could lead to cost reductions for FRP waste management in the region of 40%. The recycling label will promote waste management in order to increase the volume of FRP waste collected for recycling. It will also finance the development of new technologies and markets for FRP recyclate. Two fees make up the licence fee for the Green FRP Recycling Label:

- The recycling fee covers the recycling of parts and is based on the 'producer pays' principle and takes into account the weight of the part, the complexity of recycling (pushing for designing for recycling), and bonuses for using recyclate (to encourage and achieve recycling). The recycling fee should be added to the part price, the amount of which is variable and should be revised annually, but is around €0.06 (£0.05) per kg of end product.
- The system fee finances closing the loop, is paid by all parties involved within the composites industry, and is approximately €30 000 (£22 556) per company per year. The system fee covers the cost of the Green FRP Recycling Label system, waste management services, research and development of new and improved recycling technology, and market development activities.

There are concerns that the EU regulation on the shipment of waste within, into and out of the European Community (259/93/EEC) may have an impact on the transport of composite waste across borders to countries in which a composites recycling plant exists. EuCIA is endeavouring to ensure FRP waste is classified Green in the Green–Amber–Red list of this regulation in order to facilitate the transport of waste to existing FRP recycling plants. FRP waste for recycling is not yet listed in the Green–Amber–Red list; therefore, if it is transported between countries, it must follow the same procedure as for material on the Red list. This means that certain procedures must be followed and there is a need for notification of transport of the material, a sample for analysis before the FRP can be ground, and agreement from the origin country and all others through which it will be travelling. EuCIA argue that the individual components of FRPs are on the Green list, therefore the waste should be on the Green list.

Disposal

The most common disposal method for FRP waste is landfill as this is the easiest and currently the most cost-efficient option. Space in landfill sites is decreasing and it is thought that there are many other better uses for much of the waste that is currently sent to landfill. Legislation has been introduced to encourage diversion of waste from landfill.

The European landfill directive means that the cost of disposing of waste in landfill will increase in the near future. The directive focuses on diverting as much waste as possible away from landfill which means that the reuse, recycling and recovery of FRP waste will be encouraged over landfill.

Future and emerging technologies

Ecocomposites

Ecocomposites are made by combining wood or other natural fibres, such as flax, hemp, jute or kenaf, with natural or synthetic polymers, including polyethylene, polypropylene or polyvinyl chloride (Reinforced Plastics, 2003). They can be used to produce products for building, automotive, infrastructure and consumer applications. Ecocomposites are equally strong as usual FRPs, but are considered more environmentally friendly. Natural fibres are also attractive because they have chemically reactive surfaces which make more complete fibre–matrix bonding possible.

Ecocomposites will become increasingly common as the price of wood continues to climb, environmental problems with plastics disposal increase, and agricultural fibres pose increasingly costly disposal problems. Most ecocomposite materials can be recycled (composted or digested) or

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burned without the residues that are left with glass and carbon fibre composites (Bainbridge, 2003).

FRPs based on natural and wood fibres are one of the fastest growing markets in the plastics industry, expected to achieve double-digit annual growth rates to 2010.

Design for easier reuse and recycling

Designers need to find out what is happening to the products now to provide directions for the future. Examination of the current methods of disposal can reveal opportunities for increasing the product's recycling potential and decreasing the end-of-life costs. Typical questions are:

- Is the product typically disposed of to landfill? This can give rise to environmental impacts through the production of leachate and landfill gas and will be subject to increased costs in the future.
- Can the product be reused or recycled instead of being sent to landfill?
- If products with only minor faults are typically discarded, is it possible to salvage some of the parts or components for reuse or remanufacture?
- Is there potential for reusing modules or parts of the product at the end of its useful life?
- Does the product contain materials or components that can be easily recovered and recycled to reduce costs?
- Can various modules or parts be stamped with labels indicating their recycling potential? This is particularly relevant for plastics where standard material designations and labels are available for common materials.
- Can the product and/or its modules be designed for service or maintenance to increase their life span?
- Can a product take-back service be developed to reduce 'producer pays' costs?

Answering these questions will help to identify the existing options for product reuse, recycling or recovery. In the future these will need to be improved to minimise the total cost of the product and avoid end-of-life costs that were not accounted for in the 'first' cost of the product.

Cleaner design in the future will not be easy but the alternative high end-of-life costs are even less acceptable.

Recycled products guide

The *Recycled Products Guide* (2003) is an information resource designed to help buy recycled products. It contains the details of over 1000 companies in the UK who can supply a range of products, from recycled aggregates to kitchenware. Buying recycled products helps 'close the loop', creating a demand for the materials recovered by recycling collection schemes. For recycling to work, there needs to be markets for the products made with recycled materials. Buying recycled helps keep reusable material in the economy, reduces waste needing to be landfilled and conserves resources, particularly energy.

Summary

Existing FRP recycling plants such as ERCOM, MCR and Miljotek have proven that mechanical recycling of FRP is possible, but these plants are in need of profitable markets for the recyclate generated. In order to solve the current waste management problems, research needs to be conducted to develop new applications, write recyclate standards and develop new high-value markets for FRP recyclate. Other aspects that need addressing include reducing manufacturing waste, investigating the dismantling and collection of end-of life waste, and the construction of new recycling plants based on better technology.

The issues of recycling and sustainability continue to grow in importance in public and political spheres. Designers using FRP materials today should consider the life cycle of the application and the end-of-use properties. The issues of reuse, recycling or safe disposal of materials should be considered at the design stage.

European legislation and its transposition into national legislation pushes industry towards reuse; mechanical or chemical recycling puts limits on incineration with energy recovery and in many countries already forbids direct landfill of composite waste. Producers of materials which compete directly with FRP are very active today in promoting their material as having a functioning recycling concept.

Reuse of FRP material is possible, but is dependent on the nature of the materials used, how they are joined and fixed, and their condition. Recycling of FRP waste is a viable option, but certification schemes need to be adapted to make it easier to test components containing recycled material.

There is currently no market value for waste FRP in the UK as a recyclate. Other than a few firms which grind production waste and use it as filler, no recycling of this material takes place. Many manufacturers would like to be able to recycle their waste. However, cost appears to be the major factor determining whether this is viable.

Most production and post-service waste is currently placed in landfill. Waste disposal charges for producers remain low in comparison to the added value and cost–benefits of their products. Also, waste disposal charges are merely transferred to the end users. It appears that at present the environmental benefits of energy recovery by burning FRP in a waste incinerator are likely to be outweighed by significant disadvantages. It is thought that opportunities for reuse of FRP products will be limited. A number of examples have been found in which waste material has been successfully incorporated into a new product, but little is known about the cost-effectiveness of these recycling practices.

The development of new, high-grade markets for FRP recyclate is a high priority for the development of FRP recycling. The Green FRP Recycling Label will contribute to the development of viable waste management routes for FRP and high-value end uses for the recyclate produced.

To assist in the transition from disposal of FRP waste in landfill to recycling, industry needs to consider designing components for easier disassembly, reuse and recycling at the end of the product life.

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Further reading

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Network Group for Composites in Construction: http://www.ngcc.org.uk.
Waste Resource Action Plan (WRAP): http://www.wrap.org.uk.

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Section 8: Timber

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Chapter 60

Timber engineering: an introduction

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This chapter includes a description of the anatomy of wood, a discussion on timber properties and the factors that influence these properties, information on the testing and grading of timber, and the methods used to derive design properties. It concludes with a short discussion on timber as a sustainable material.

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Introduction

Wood is a natural organic material that has been used for many centuries for the construction of buildings, bridges and a variety of other structures. It remains an important construction material today as research and improved technology have led to a better knowledge of the material behaviour. This has helped designers to use timber more efficiently and safely and in more challenging and exciting applications.

Timber has many important attributes. It has a warm texture and attractive appearance and is often used for internal finishing as well for the main structure (**Figure 1**). It is easy to work with and can be produced in a wide range of shapes and sizes. It has a high strength-to-weight ratio and has good thermal insulation properties. Timber can be used compositely with concrete and steel. It is the only construction material that does not contribute to greenhouse emissions and is a fully renewable and largely recyclable material.

Because it is naturally grown, timber is a complex building material. Its properties are highly variable and are sensitive to environmental and loading conditions. It is a highly anisotropic material with high strength and stiffness parallel to the grain but low properties perpendicular to the grain. These factors must be taken into account in the design of timber structures.

In this chapter, the behaviour and properties of timber are discussed. Starting with an examination of the microstructure of wood, the properties of wood and the factors that influence them are elaborated. The performance of structural timber is influenced not only by the basic wood properties but also by the presence of 'defects' such as knots. The influence of these defects on the structural properties is examined. Testing and grading is used to classify timber into different grades or strength classes to satisfy different end user requirements and ensure product reliability. Methods of testing and grading are described together with procedures for determining characteristic values for design purposes. Finally, the environmental impact of timber construction is considered.

Basic properties of wood

Tree species are normally grouped into two categories, namely softwoods and hardwoods. The softwood species are coniferous and include spruces, pines, and firs. These trees generally retain their needle-like leaves throughout the year. The hardwood species are deciduous and include oaks, birches, and maples. They have broad leaves, which they lose in winter. The wood from both species is used for structural timber, but softwoods are much more common in Europe due to their greater availability and lower cost.

The cross-section of a tree trunk has a number of important features, as shown in **Figure 2**. Working from the outside in, these are the bark, the cambium, the sapwood, the heartwood and finally the core or pith. The cambium is where new wood cells are formed. The younger outer part of the trunk, or sapwood, is used to store nutrients and to transport them along the axis of the tree. The width of the sapwood band can vary from about 25 mm to about 150 mm. The main function of the older heartwood is to provide structural support for the tree. There is no difference in the mechanical performance of sapwood and heartwood. Sapwood is, however, more susceptible to insect attack due to the presence of nutrients. The inner 5 to 20 growth rings are known as juvenile wood and this usually displays greater longitudinal shrinkage than more mature wood.

The trunk cross-section in both the sapwood and heartwood is characterised by distinctive annual growth rings. These occur because the wood formed during the early part of the season when the growth is more active (earlywood) is less dense and therefore lighter in colour than that produced later in the season (latewood). Under conditions of rapid growth, the growth rings are wider resulting in wood of lower density and strength. For this reason, ring width is sometimes used as an indicator of the strength of timber.

The majority of wood cells is tubular in section and elongated. They are called fibres or tracheids and are about 2 to 5 mm long and 10 to 50 μm wide. Cell wall thicknesses for

Figure 1 Coillte headquarters, Ireland (courtesy of Coillte)

softwoods are in the range 2 to 5 μm. They are aligned parallel to the axis or trunk of the tree. This is referred to as the grain direction. The individual cells are glued together by a lignin-rich layer called the middle lamella.

The cell wall itself has a multi-layered structure which gives the wood its strength and stiffness. Each layer comprises cellulose microfibrils embedded in a matrix of lignin and hemicellulose with the microfibrils oriented differently in the different layers. The thickest layer has microfibrils oriented parallel to the cell axis and as a result is very effective in carrying tension loads. The other layers have microfibril angles of between 50–70° and these layers help prevent buckling of the cell wall under compressive loading. In softwoods, the central core of the cell, or lumen, is used to transport nutrients. In hardwoods, the nutrient transport is by means of special large-diameter tubular cells called vessels embedded in the fibres. Hardwood fibres generally have smaller lumen diameters and thicker cell walls than softwoods.

The arrangement of wood cells or fibres parallel to the trunk is very efficient in carrying the actions sustained by the living tree. These are primarily axial compression forces due to self-weight and bending moments due to wind loading. Wood is very strong in tension and compression parallel to the grain direction. However, the properties perpendicular to the grain direction tend to be poor. The actual strength values vary significantly not only between species but also between individual trees within a species and indeed within an individual tree. The differences between trees can result from different environmental conditions and forestry practices. Within an individual tree, the difference is due not only to local density variations but also to the presence of knots, spiral grain and reaction wood.

Non-isotropic characteristics

The properties of wood are strongly influenced by direction because of the arrangement of its fibres and the orientation of the microfibrils in the cell walls. In particular, wood is orthotropic as three orthogonal directions of symmetry can be identified. The longitudinal (L) direction, or the direction parallel to grain, is aligned with the axis of the tree trunk. The radial (R) direction lies along the radius of the tree cross-section while the tangential (T) direction is tangential to the growth rings (**Figure 3**). The radial and tangential directions are referred to as directions perpendicular to grain. The wood strength, modulus of elasticity and other characteristics such as shrinking and swelling differ in the three directions. The load-carrying capacity in the direction parallel to grain is significantly greater than in the perpendicular direction. The dimensional stability or resistance to distortion under fluctuating moisture content is also greater. Structural timber is normally sawn into boards with the long axis of the board aligned parallel to grain.

Moisture and wood

Wood is a hygroscopic material in that it exchanges moisture with its surroundings and its properties vary significantly with moisture content. The moisture content in wood is defined as the ratio of the mass of water that can be removed from the wood to the mass of the dry

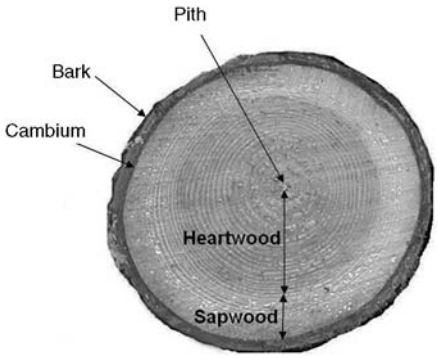

Figure 2 Cross-section of tree trunk

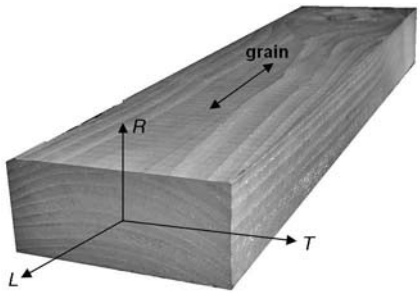

Figure 3 Orthotropic directions for wood properties

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wood. In a growing tree the moisture content is highly variable and can be as high as 200%. This moisture is present as free water in the cell cavities or lumens and as bound water in the cell walls. When wood is harvested and processed, the wood dries from the green state to a moisture content that is in equilibrium with the relative humidity and temperature of the surroundings. This is known as the equilibrium moisture content (EMC).

As the wood dries, the free water in the cell cavities is lost first. After this the bound water in the cell walls is lost. The moisture content at which all of the free water has been removed and the cell walls are still saturated is known as the fibre saturation point (FSP). For most species, the FSP is in the range 25–35%. An average value of 30% is normally assumed. The physical and mechanical properties of the wood change significantly as the moisture content is reduced below the FSP, with strength and stiffness values increasing with decreasing moisture content. Above the FSP most properties remain constant.

Changes in moisture content below the FSP also result in swelling and shrinkage of the wood. If these moisture-induced deformations are prevented in a structure, stresses will develop. For this reason, it is important that, before installation, structural timber is dried to a moisture content close to that which it will experience in service – normally its EMC. When timber is used in an environment where there are wide fluctuations in relative humidity and temperature, significant moisture-induced stresses can develop, particularly in directions perpendicular to grain.

At a standard indoor climate with a temperature of 20°C and a relative humidity of 65%, the equilibrium moisture content of softwoods is about 12%. This is the accepted reference moisture content at which testing of wood is performed and at which international codes and standards generally specify design values.

The moisture content of wood can be measured by oven drying or by use of moisture meters. These meters operate on the principle that the moisture content influences the electrical resistance and are calibrated for individual species.

Physical properties of wood

Density

The density of wood is determined mainly by the amount of wood substance per unit volume and the moisture content. The higher the proportion of wood substance the greater the density and also the higher the mechanical properties. Mean values for the density of softwoods and hardwoods range from about 400 to 650 kg/m^3 and 500 to 1200 kg/m^3, respectively. Wood of high density tends to shrink and swell more with changes in moisture content than wood of low density.

Shrinking and swelling

Below the fibre saturation point, wood shrinks and swells as its moisture content falls and rises. The rate of shrinkage/swelling varies with direction. Values in the parallel to grain direction are 5–10% of those in the perpendicular to grain direction while values in the tangential direction can be 1 to 2 times those in the radial direction. Because of these differences in shrinkage rate, cross-sections can distort during drying. Splits can occur in large cross-sections when the drying rate is too high. In the region of knots, the grain direction is distorted which can result in warping of the specimen during drying. The tangential shrinkage of softwoods such as spruce can be as much as 8% when dried from 30% to 0% moisture contents. Compression wood and juvenile wood often display high levels of longitudinal shrinkage resulting in bow, spring and twist (**Figure 4**). In the presence of spiral grain, twist occurs during moisture changes. Differential movements in the radial and tangential directions result in cupping (**Figure 4**). Limits on the level of acceptable distortion are given in grading rules for structural timber.

Thermal properties

Timber has low thermal conductivities and coefficients of thermal expansion (CTE). The degree of anisotropy in thermal properties is less than in structural properties. The thermal conductivity parallel to grain is about 2 to 3 times that perpendicular to grain. The average value for softwoods is 1.2 W/mK. The CTE parallel to grain is about 10–20% of the perpendicular to grain value and is typically of the order of 3.5×10^{-6}/°C.

Durability

Wood is susceptible to attack by a variety of organisms. Fungi, which use the wood as a source of food, cause decay that can result in a reduction in mechanical properties. These fungi require moist conditions for growth and can be prevented by ensuring that the moisture content of the wood is kept below 20%. A number of insects and borers attack wood and destroy it, including termites or white ants. In many cases, keeping wood dry will prevent damage from their attack so it is important in designing with wood to minimise the absorption of water by wood. If the moisture content cannot be controlled, then the timber should be treated with preservative or a species of timber with a high level of natural durability should be used. Application of preservatives is carried out using pressure to ensure adequate penetration. Commonly used preservatives are organic solvent or micro-emulsion preservatives, such as ACQ, copper triazole and borates. Chromated copper arsenate (CCA), which was one of the most common wood preservatives, is now banned for use in residential construction in many countries because of

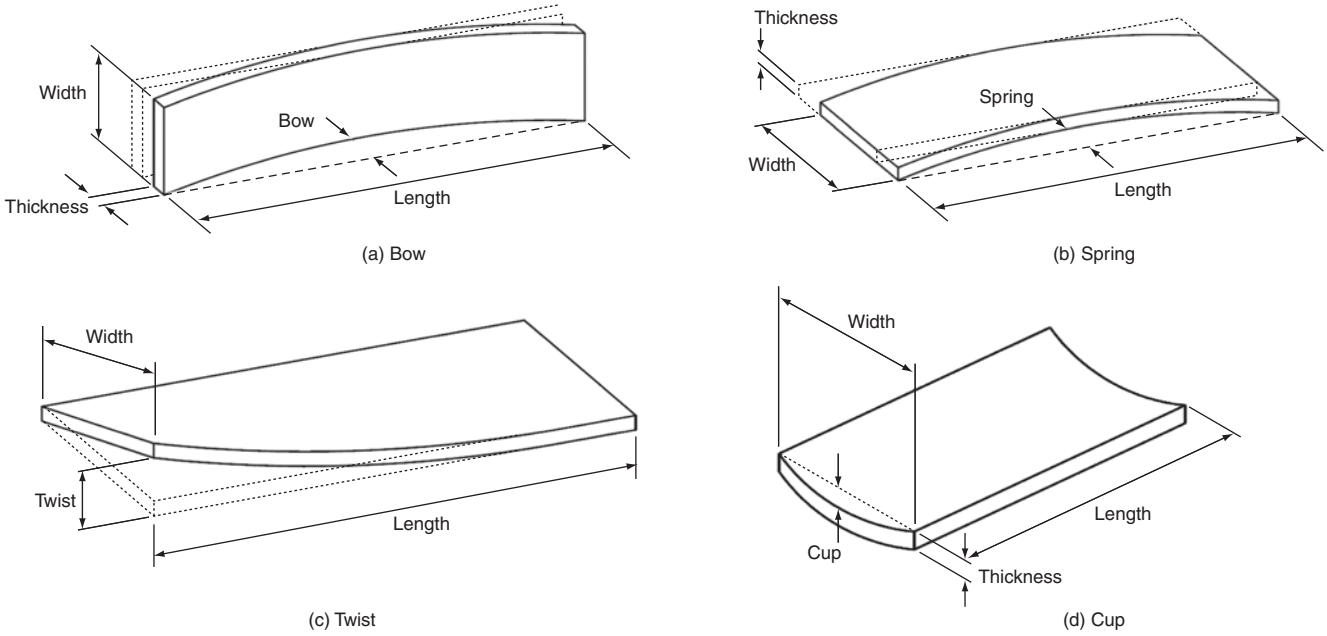

Figure 4 Geometrical imperfections and distortion in timber

health concerns. The selection of the appropriate preservative is based on the hazard level to which the timber will be exposed during service. It is very difficult to quantify the level of strength reduction due to decay or insect attack and for this reason they are prohibited in wood for structural applications.

Mechanical properties of wood

Parallel-to-grain properties

The parallel-to-grain direction is the strongest and stiffest from a structural perspective. The tensile strength of softwoods parallel to grain at 12% moisture content generally ranges between 70 and 140 MPa. The compression strength is lower and is usually in the range 30–60 MPa. For hardwoods, these values are generally higher. These values are for clear, straight-grained wood samples. For structural timber, the effects of knots and other strength-reducing characteristics (commonly referred to as defects) can reduce these values by a factor of as much as 10. The failure mode is load dependent with tension failure being brittle while compression failure is ductile. The modulus of elasticity in both tension and compression is generally taken to be the same. At 12% moisture content, typical values for both softwoods and hardwood are in the range 7–14 GPa.

Perpendicular-to-grain properties

Perpendicular-to-grain properties are significantly lower than the equivalent parallel-to-grain properties. The tensile strength in the radial and tangential directions can be as low as 5–8% and 3–5%, respectively, of the values in the grain direction. Because of these low strength values, it is advisable to design timber structures in such a way that tension stresses perpendicular to grain do not occur or are minimised. Situations where tension perpendicular to grain stresses arise include at joints, in tapered or curved members and in notched beams. Compressive strengths perpendicular to grain are approximately 10–20% of the parallel-to-grain values. In compressive failure, the cells flatten and the cells walls touch resulting in a densification of the wood and increasing compressive resistance. However, this is also accompanied by large deformations which can occur even at relatively low load levels.

Engineered wood products

Research into finding ways of making more efficient use of timber as a structural material has led to development of a wide range of engineered wood products. These include panel products such as oriented strand board (OSB) and plywood, structural elements such as glued-laminated timber (glulam), laminated veneer lumber (sometimes referred to as Kerto) and I-joists. These products are made from wood strands, flakes, veneers or boards that are bonded together under heat and pressure generally with their grain angle parallel to the principal axis of the element formed. A major advantage of the manufacturing process is that defects are distributed throughout the element resulting in greater product reliability, strength, stiffness and dimensional stability compared to an equivalent solid timber element. Elements with greater depths and spans compared with solid wood can be easily produced. These products utilise wood that would otherwise be unsuitable for use in the construction industry such as fast-growing small-diameter trees and low-grade wood material. This relieves some of the demand on mature high grade wood material.

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The modulus of elasticity values perpendicular to grain are approximately equal in the radial and tangential directions and are about 4–9% of the parallel-to-grain values for softwoods.

Flexure and shear

The most commonly referenced strength property of wood is its modulus of rupture. This is defined as the bending stress in a flexural member at the failure load and is computed assuming an elastic stress distribution. For clear or defect-free wood, this is lower than the tensile strength parallel to grain. The converse is true for structural timber, which normally contains large numbers of strength-reducing characteristics or 'defects'. Research has shown that size effects are important in flexural members as the larger the member the greater the probability of having a large defect which will reduce strength.

The shear strength is characterised by three types of failure, namely shear parallel to grain, shear perpendicular to grain and rolling shear. In shear parallel to grain, the failure mode is quasi-brittle with shear strengths of about 5–12 MPa.

Joints

As with all structures, the structural capacity of timber structures is often limited by the joint capacity. A number of different joint configurations are used, including those made with mechanical fasteners/connectors, adhesives and traditional carpentry techniques.

Dowel-type fasteners are the most common form of mechanical fastener and include nails, screws, lag screws, dowels, drift pins, drift bolts and bolts. Nails are used for light loads and are commonly used for diaphragms, shear walls and trusses. Screws are better than nails in resisting vibratory or pull-out loads. Bolts, dowels and special timber connectors such as split-rings and shear plates are used to transfer heavy loads. Punched metal plate connectors are used in roof truss construction.

Structural adhesives in timber are commonly used to produce larger sections and longer members. Finger jointing and scarf jointing are used to increase the length of a member but finger jointing is also used to form corner joints. Glued laminated timber beams, I-joist and box-section beams are generally assembled using glued joints. Structural adhesives are also used in hybrid joints which also incorporate steel or fibre-reinforced plastic rods or plates.

The use of traditional carpentry joints in large structural elements is becoming feasible with the wider availability of CNC machines for use in offsite manufacture.

Factors that influence mechanical behaviour

Strength-reducing characteristics or naturally occurring defects

The processing of logs into structural timber results in a disruption of the structure of the naturally grown wood. This timber is referred to as in-grade timber and it has inferior properties to clear wood. This is due to the presence of strength-reducing characteristics or defects such as knots, slope of grain, gum veins, reaction wood, etc. Because of this, reliable mechanical properties of timber cannot be directly derived from the clear wood properties but must be determined by testing of in-grade specimens according to standardised procedures. In addition to a reduction in strength, the behaviour of structural timber is considerably more variable than clear wood as the pattern and frequency of defects varies from one piece to the next. Due to the random distribution of defects, the strength of a piece of timber depends on the size of the piece and the way in which it is loaded.

Knots occur where branches joined the main stem of the tree and formed very efficient joints. They affect the strength due to the fact that the fibre orientations in the knot are generally perpendicular to the fibre direction of the member, and the grain of the wood surrounding the knot is also severely distorted. The strength-reducing effect depends both on the size of knot and its location. The larger the knot, the greater will be the grain deviation and resulting loss of strength. Similarly, a higher frequency of knot occurrence will generally result in a lower strength than timber with fewer knots. Location of these characteristics is important: edge knots in a member subject to bending are more detrimental than the same sized knot located at the neutral axis. Knots can be present in large numbers in timber members (particularly softwoods) and their effect in degrading of strength properties is significant. In tension or bending tests of timber members, failure almost always occurs at the location of a knot (in the tension zone for bending members).

Sloping grain or cross-grain occurs when the fibre direction is not parallel to the axis of the structural member. This may occur for a number of reasons. It may be due to the way the piece is cut from the log or alternatively, the fibre direction may not have been parallel to the axis of the log. Spiral grain is an anomaly whereby the wood fibres grow in a spiral direction around the tree axis instead of parallel to it. Sloping grain reduces the strength of the timber because the stress parallel to the axis has components parallel and perpendicular to the grain direction. Since the strength perpendicular to grain is very low, this limits the strength of the board. The strength reduction increases with increasing slope of grain. Another consequence of sloping grain is the tendency of pieces to warp with changes in moisture content.

Reaction wood is abnormal wood which may develop, for example, in a young tree leaning from the vertical due to wind, or it may occur in branches. In softwoods, the abnormal wood is called compression wood as it occurs in areas of high compressive stress. Reaction wood in hardwoods develops in regions of high tension and is called tension wood. Reaction wood has higher density than

normal wood and may be stronger. However, it displays high levels of dimensional instability with changes in moisture content and is therefore not used in structural members.

Fissures, in the form of shakes, checks and splits, may arise due to stresses developed during drying of the timber, due to mechanical loading or may occur during the growth of the tree. Their main effect is a reduction in the shear strength of the member. They can also act as a pathway for moisture to penetrate into the interior of the members, which may result in decay.

Moisture

The mechanical properties of clear wood can be assumed to vary linearly with moisture content below the FSP and the influence of moisture content on the properties of timber varies with respect to the property being considered. The tensile strength is slightly reduced with increasing moisture content, with the higher grades most affected. The bending strength is also reduced with increasing moisture content and the failure mode in bending is moisture dependent. At low moisture contents, bending failures occur in the tensile zone, whereas at high moisture contents failures tend to occur in the compression zone. Design codes around the world include stress modification factors to account for the influence of moisture.

Duration of load

The duration of load has a significant impact on both strength and stiffness. For a given magnitude of load, the strength of a timber member is reduced as the duration of the load increases. This loss of strength may be as high as 40%, which basically means the long-term strength for permanent loads such as self-weight or dead loads is only about 60% of that for the timber when it is first loaded in a structure. On the other hand, the duration of load effect on strength is less and the load-carrying capacity is higher for members subjected to rapidly applied and very short-term loading, such as peak wind events.

Significant creep deformations are also experienced under sustained load. Under constant climatic conditions, the creep deformation is approximately the same as the instantaneous elastic deformation for seasoned timber and can be two to three times the instantaneous deformation for unseasoned timber. Changes in moisture content have a major influence on the duration of load behaviour. As the moisture content is increased, the time to failure for a particular stress level is reduced and creep effects are much greater. Moisture cycling together with mechanical loading results in what is known as the mechano-sorptive effect. If wood is loaded in the unseasoned (green) state and subjected to varying climatic conditions, creep deformations of up to five times the initial deformation may result during the drying process.

Cyclic loads

Strength values for members subject to repeated loading should be reduced by about 50% for static clear samples where small knots are present and by 70% where knots and sloping grain is present.

Temperature and fire

In the normal range of ambient temperatures, the strength of wood does not vary, whilst at higher temperatures strength values are generally reduced. If the exposure time to higher temperatures is short, strength recovery is possible. However, for longer exposures permanent damage generally will result depending upon both the temperature and duration of exposure.

Wood is a combustible material and will ignite when exposed to fire. Light structural members are normally protected from fire by using a fire-resistant cladding, such as plasterboard. Large structural members have good inherent fire resistance because a char is formed on the exposed surface that acts as an insulator and the wood inside the char experiences only a small increase in temperature. The rate of charring is almost constant and is about 0.5 mm/minute for softwoods and 0.67 mm/minute for hardwoods. The strength and stiffness of the unburnt wood remain unchanged and the capacity of the section after a period of fire can be determined from the residual cross-section. Fire protection can be provided by using intumescent coatings or by impregnating wood with flame- or fire-retardant salts, which raise the energy level required to cause wood to burn.

Measurement of mechanical properties

Standard test methods are available to determine the mechanical properties of timber. These tests are normally performed on timber that has been conditioned to about 12% moisture content. The size of the specimens and the rate of loading are prescribed. In Europe, testing is usually performed to the standard BS EN 408: 2003.

Bending properties are normally determined by symmetrically loading a specimen at the third points of a simply supported span of 18 times the specimen depth. This method is referred to as a 'four-point' bending test and the intent is to create a zone of constant moment with no shear in the most highly stressed mid third of the beam under test. The modulus of elasticity is determined from the slope of the load–midspan deflection curve and the bending strength is determined from the maximum load and the section modulus. This same test set-up may also be used to determine the shear modulus.

Tension and compression parallel to grain properties are found by applying an axial force to a test piece having a full-size cross-section and sufficiently long to provide a test length clear of the grips. For perpendicular-to-grain

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tests, the test piece dimensions are prescribed. In all cases, the deformation is measured over a central gauge length. The modulus of elasticity is determined from the slope of the load–deformation curve. The tension or compression strength is found by dividing the failure load by the cross-sectional area.

Characteristic values

Due to the presence of random defects, the testing of samples from a population will result in mechanical properties that can be represented by a statistical distribution. Limit state design codes are based on characteristic values of these properties. Characteristic values are determined as the weighted means of the sample lower 5-percentiles for strength properties and density, whereas the weighted mean of the sample averages (50-percentile) is used for determining modulus of elasticity. Representative samples from a population, defined in terms of species, source, stress grade and manufacturing process, are tested in accordance with standard procedures. For each sample, the 5-percentile strength value, f_{05}, is found by ranking all the test values for a sample in ascending order and finding the value below which 5% of the values fall. A larger test sample will usually give a much more reliable estimation of the 5-percentile value and a variety of statistical methods are employed to fit the data distribution, with a log normal analysis being one of the more popular that is used.

The 5-percentile density values are often determined from assuming a normal distribution with a coefficient of variation of 10% as follows:

$$\rho_{05} = \rho_{mean} - 1.65(0.1\rho_{mean}) = 0.84\rho_{mean}$$

where ρ_{mean} is the mean density.

Characteristic values are adjusted for sample size and variability and for any deviation to the standard test conditions, such as sample dimensions and moisture content (BS EN 384: 2004).

Strength classes

The European strength class system assigns structural timber to grades or strength classes with defined properties. It comprises 12 classes for poplar and softwood species, namely C14, C16, C18, C20, C22, C24, C27, C30, C35, C40, C45 and C50, and six classes for hardwood species, namely D30, D35, D40, D50, D60 and D70.

The letters C and D refer, respectively, to coniferous and deciduous while the numerical values represent the characteristic bending strength in MPa for the class. Timber populations are assigned to a strength class if the characteristic values of bending strength and density of the population are equal to or greater than the values for the strength class and if the characteristic mean modulus of elasticity in bending equals or exceeds 95% of the value for the strength class. The European Standard BS EN 338: 2003 defines the characteristic strength $f_{m,k}$, stiffness $E_{0,mean}$ and density ρ_k values for each strength class, a selection of which is given in **Table 1**.

Property		C16	C18	C24	C30	C35	C40	D35	D50	D60
Bending strength: MPa	$f_{m,k}$	16	18	24	30	35	40	35	50	60
Mean MOE parallel: GPa	$E_{0,mean}$	8	9	11	12	13	14	10	14	17
Density: kg/m^3	ρ_k	310	320	350	380	400	420	560	650	700

MOE, modulus of elasticity

Table 1 Characteristic properties for selected strength classes

Research has shown that all important characteristic strength and stiffness properties can be calculated from either bending strength, modulus of elasticity or density. For any strength class, the characteristic values of other mechanical properties can be found using the following relationships.

Tensile strength parallel to grain

$$f_{t,0,k} = 0.6f_{m,k}$$

Compression strength parallel to grain

$$f_{c,0,k} = 5(f_{m,k})^{0.45}$$

Shear strength

$$f_{v,k} = \min[3.8\,\text{MPa}: 0.2(f_{m,k})^{0.8}]$$

Tensile strength perpendicular to grain

$$f_{t,90,k} = \min[0.6\,\text{MPa}: 0.0015\rho_k]$$

Compressive strength perpendicular to grain

$$f_{c,90,k} = 0.007\rho_k \text{ for softwoods}$$
$$= 0.015\rho_k \text{ for hardwoods}$$

Modulus of elasticity parallel to grain

$$E_{0.05} = 0.67E_{0,mean} \text{ for softwoods}$$
$$= 0.84E_{0,mean} \text{ for hardwoods}$$

Mean modulus of elasticity perpendicular to grain

$$E_{90,mean} = E_{0,mean}/30 \text{ for softwoods}$$
$$= E_{0,mean}/15 \text{ for hardwoods}$$

Mean shear modulus

$$G_{mean} = E_{0,mean}/16$$

Grading of timber

The grading of timber for structural purposes is essential for ensuring that the material is reliable and has defined properties that can be used in design. As the timber cannot be tested to failure during a grading process, non-destructive evaluation procedures are used. This grading procedure is essentially a sorting process that assigns each individual piece of timber to a strength grade/class by defining grade limits for characteristics (known as grading parameters) that are correlated to the strength of the

timber. In Europe the grade limits are related to a reference moisture content of 20%. Two main methods of grading are currently in use, namely visual grading and machine grading. BS EN 14081: 2005 is the European standard for strength grading of rectangular structural members.

Visual stress grading is the oldest form of strength grading. It involves carrying out a visual inspection of the strength-reducing defects in a board. Different grading rules have developed in different countries with different numbers of grades, grade limits and ways of measuring the grading characteristics. However, all national visual grading standards must comply with the general requirements specified in BS EN 14081-1: 2005. In the UK, visual grading of softwoods and hardwoods is carried out to BS 4978: 2007 and BS 5756: 2007, respectively.

The grading process involves a visual examination of the four faces of each piece. The following characteristics are assessed: knots, slope of grain, rate of growth, wane, fissures, resin and bark pockets and distortion. The codes give limits for these characteristics for assignment to a particular grade. There are two softwood grades, namely 'general structural' (GS) and 'special structural' (SS), one tropical hardwood grade, 'structural tropical hardwood' (HS) and four temperature hardwood grades, 'heavy structural temperate hardwood' (THA and THB) and 'general structural temperate hardwood' (TH1 and TH2). Any piece containing abnormal defects such as compression wood, insect damage, fungal decay, damage, combination of knots and/or other characteristics that could result in a significant decrease in strength resulting in a lack of serviceability, is excluded from the grades. Timber that has been visually graded to visual grading standards can be assigned to a strength class. The grade assignment depends not only on the visual grade but also on the species of timber and its source.

Machine stress grading operates on the principle that the strength of the timber is strongly correlated with one or more properties of the timber that can be measured by the machine – most commonly, minor axis deflection. By setting limits to these properties, timber is graded directly to a strength class. Most grading machines currently in use are designed to bend each piece of timber over a short span about its minor axis as it passes though the machine at high speed. The machine determines the bending modulus of elasticity of the timber at regular intervals along the length of the piece in one of two ways: the deflection induced by a fixed load or the load required to induce a fixed deflection are measured. Newer methods of determining the modulus of elasticity of the piece of timber use methods such as vibration, ultrasound and microwave techniques. These methods ensure that the timber is not damaged during the grading process. Machines that measure several grading parameters can give better accuracy. A combination of modulus of elasticity and knot parameters is better correlated with strength than modulus of elasticity alone. Optical scanners can be mounted on the grading machine to detect the presence of knots. These higher-efficiency machines are more expensive than bending-only machines and therefore are not as widely used at the present time.

Machine grading is carried out using one of two systems known as 'machine controlled' and 'output controlled'. The output controlled system is used for grading large volumes of a limited number of sizes, species and grades. This system involves destructive tests on specimens of the daily output. The machine-controlled system was developed in Europe because of the wide variety of sizes, species, and grades that are produced. This system does not require destructive testing of the timber but instead involves strict assessment and control of the grading machines and the derivation of machine settings for different species, sources and grades. Further details of grading processes are discussed in Chapter 2.

Sustainability

Wood is a natural resource capable of being grown in most parts of the world. Implementation of proper management of a forest resource means that, at least in theory, it is possible to grow an endless supply of timber. The environmental benefits of wood are immense. Trees absorb CO_2 from the atmosphere, store the carbon and release O_2 to the atmosphere, and so wood forms a carbon sink – which is unique for an engineering material. Timber harvested from trees and used in construction continues to store carbon. The carbon is only released back into the atmosphere when the wood decays or is burned. This situation is carbon neutral as it releases the carbon originally absorbed from the atmosphere during photosynthesis. At the end of life of a timber structure, a certain percentage of the material can be recycled for direct reuse in construction or in the manufacture of panel products. Wood waste arising from harvesting and processing as well as from construction and demolition of buildings can be used as a replacement for fossil fuels. The development of engineered wood products has led to maximisation of the use of the material and a reduction in resource use and wastage.

The processing and transportation of timber requires little energy compared to the manufacture of steel and concrete. This embodied energy is given in **Table 2** for the three materials. As the embodied energy is expressed in

Material	Wood	Concrete	Steel
Embodied energy: MJ/kg	1.2	1.2	32.0
Emission: kgCO_2/kg	0.0000	0.0194	0.5168

Table 2 Embodied energy and CO_2 emissions in construction materials (adapted from Gonzalez and Navarro, 2006)

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MJ/kg and is therefore a function of the material density, care should be taken in interpreting these data. As the densities of steel, concrete and timber are approximately 7800 kg/m^3, 2400 kg/m^3, and 500 kg/m^3, respectively, the actual differences in embodied energy are very significant. In addition, wood is a good insulator and as a result the energy required for heating and cooling of timber buildings is reduced and this can result in significant savings in operating costs over the life of a building. A number of comparative studies of residential construction have shown that timber elements have less global warming emissions than steel or concrete, as shown in **Table 2**.

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Chapter 61

Timber products and manufacturing processes

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Timber products are a green and renewable natural building material. The demand for these products will keep increasing in the future. This chapter presents a brief discussion on some of the most common timber products (round wood, sawn timber and engineered wood products), their manufacturing processes and their scope of application.

Introduction

Timber is one of the oldest and the only renewable natural building material in the world. It was the primary structural material until the beginning of the twentieth century. Since then more modern building materials, such as steel and concrete, have been used to allow long-span and tall structures to be built. However, the demand for structural timber and wood products keeps increasing as a building material for single and multi-family housing and low-rise commercial buildings. For example, over 90% of residential housing in North America and Japan is built with wood frame or post-and-beam systems where over 2 million housing units are built annually. Wood is more suitable than the other building materials in these applications for many reasons, some of which include cost, ease of workmanship, customer preference for natural materials, and thermal insulation properties of wood. With issues of sustainability coming into the focus of the general public, the demand for green building material and technology will increase in the future to further fuel the demand for wood products.

The sustainable commercial use of the forest resource must be balanced against other demands, with recognition of the impact of the forest ecosystem on the environmental quality of the world and the need to preserve old growth forests as parks and recreational areas. The supply of high-quality old-growth timber suitable for engineered structural uses is limited by reduced access to supplies of old-growth timber. This has been an important driver for improving the efficiency of wood fibre utilisation from the forest to the final product that offers a viable solution to the changing nature of timber supply. This includes the development of new technologies to improve the harvesting and production process to reduce material losses. Also from the utilisation perspective, as it is relatively easy to cut and shape wood, new technologies and new products are being developed that can lead to the commercial use of smaller-diameter, lower-quality logs and previously underutilised species and residues. This chapter provides a brief overview of common timber products, their method of production and scope of applications.

Round wood

Round wood in the form of large-diameter logs has had widespread historical use in construction. These buildings can be built as either solid log houses or post-and-beam construction and typically have aesthetically pleasing rustic architecture and fit very well in rural and suburban settings including resorts, residences and recreational homes. In the form of solid log houses, the walls have the advantages of the availability of large thermal mass of the logs, creating one of the best forms of insulation against either heat or cold. Also, the thick round wood used in the solid wall element can provide considerable fire resistance, allowing the structure to maintain its integrity for a specified amount of time because fire rating is dependent on the thickness of the timber element. Log post and beam construction using large-diameter logs for vertical columns to support horizontal beams is also commonly used. Here the connections between the various elements require carefully engineered details.

There are very few standards or specifications available on the log quality for use in these types of construction. Logs consisting of sound wood must be debarked to reduce the chances of attack by insects and decay caused by fungi. Other criteria may include a minimum diameter of the logs, green (unseasoned) versus dry (seasoned) logs, and the presence and direction of spiral grain in the logs. When using unseasoned material, care must be taken to account for shrinkage and to minimise the introduction of shrinkage stresses when the material dries out in-service. Various softwood species are used for log homes, including cedar, pine, spruce and fir.

Timber or lumber

Timber or lumber are sawn wood products intended for use in a wide range of structural applications including wall

studs, top and sill plates in wall frames, floor joists, planks, header beams (sometimes known as lintels), roof rafters, chord and web members in trusses, as well as post and beams members. Softwood or hardwood logs from the forest are brought into sawmills for processing. The logs are cut to length and then debarked before undergoing the primary breakdown process using head-rig equipment. In modern mills this equipment has the benefit of advanced laser scanning equipment to estimate the shape and geometry of each log. Based on the estimated shape and geometry, optimisation software is used to calculate the position and orientation of each log, which can optimise the recoverable volume of sawn timber from the log. This information is established at line speed to control the sawing equipment to rotate and position each individual log for high-speed sawing. Typically, either twin band-mills (two blades) or quad band-mills (four blades) are available in the primary breakdown equipment. This stage of processing is critical to the successful operation of the mill, as a slightly misplaced saw line at this stage can dramatically reduce the potential yield of a log.

The primary breakdown stage cuts the logs into centre cant and side boards (flitches). A cant is a large four-sided timber not yet to cut to final size. This material is ready for further processing in a secondary breakdown stage where it is resawn using a rotary gang edger system with multiple circular saws to simultaneously cut the cant into several boards in a single pass. In some mills, curve saws can be used to account for the lengthwise bend of the logs (sweep) to increase yield. Also the mills may use thin guided saws to keep saw kerfs in the range 3.0–3.3 mm. More advanced mills with good control and equipment maintenance systems can further reduce the saw kerf to improve yield.

Throughout the sawing process, inspection stations are provided to remove material deemed unsuitable for further sawing. The removed material is typically sent to a chipper to produce chips needed as raw material for pulp and paper production. Trimming to length is the final step of the sawing process where scanners can be used to guide trimmers to cut off defects near the end of the board to further increase its value.

The type of logs (size/species) available to the mill and the type of products produced by the mill dictate the type of equipment installed in the mills. For example, mills in interior British Columbia, Canada, are built for high-speed and large-volume processing of the spruce–pine–fir species group. The logs from these trees tend to be smaller in diameter compared with the species available along the coastal region of British Columbia. Therefore, the type of processing equipment and optimisation strategies are different for the mills in these two regions.

After sawing, the material is dried in a kiln to, typically, an average moisture content of 18%. Some mills in the coastal region of British Columbia and western US produce green (unseasoned) timber. The green material is produced oversized and intended to dry to a target moisture content and shrink to target size during the construction process. Many of the mills in coastal British Columbia are considering or actually converting to producing a kiln-dried product as overseas customers are demanding dried products. After drying, the timber is then processed through high-speed planers that operate at speed of 5 m/s.

Visually stress-graded material

The concept of grading was introduced in chapter 'Timber engineering: an Introduction' as a process that sorts timber into groups or grades of timber which have similar mechanical properties that are linked to characteristic properties that can be used for design purposes. While much of the timber produced in Europe uses machine grading, in North America most sawn timber used in construction is sorted into grades based on visual inspection of certified graders. The graders inspect the four faces of each piece and look for the presence of natural features such as knots (size and location), slope of grain, checks, splits, wane (lack of wood), warp, stain and manufacturing defects (e.g. saw cut). Based on the quantity and severity of observed defects, the grader assigns a grade on the piece. The North American grades for structural light frame application are designated as Select Structural, No. 1, No. 2 and No. 3. A grade stamp is applied on each piece to identify the grade, mill, species and grading agency. The grading rules are established in national standards maintained by standard bodies. Inspection by third-party grading agencies is periodically carried out to check for grading accuracy. Although an individual piece has a grade assigned to it, the visually graded material is usually packaged and shipped with different grades mixed together in a package. For example, the Select Structural, No. 1 and No. 2 material are shipped as No. 2 and better material. Either the users take the effort to re-sort the material or they use the design properties of the lowest grade in the design process.

Machine stress-graded material

An alternative method that is widely used in many parts of the world for grading sawn timber is the machine stress rating. This method is based on the well-known principle that there is a positive correlation between the stiffness and strength of wood. This relationship is not very strong, with a typical coefficient of determination r^2 between bending strength and bending modulus of elasticity of approximately 0.4 to 0.5. This means that each grade will typically have a fairly narrow range of modulus of elasticity but the range (variability) of associated bending strength can still be rather large. Furthermore, other strength properties such as tension and compression strengths parallel to grain, tension and compression strengths perpendicular to grain,

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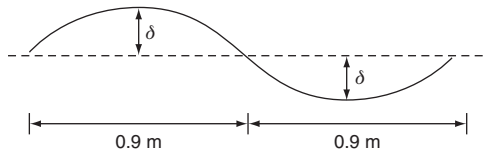

Figure 1 Schematic of a typical MSR machine

and shear strength are also inferred from the estimation of modulus of elasticity, as described in Chapter 1. Nevertheless, the machine stress rating process tends to allow a more consistent product to be produced when compared with visually graded material and therefore it often commands a slight price premium.

Typically, individual pieces of kiln-dried and planed timber are fed lengthwise through the machine stress-grading equipment (MSR machine). The machine operates at planer speed and each piece undergoes flat-wise (minor axis) bending through a set of rollers into a prescribed S-shape. The span of each side of the 'S-curve' is approximately 0.9 m and the deflection in midspan of each $\frac{1}{2}$ curve (δ) is approximately 5 mm, as shown in **Figure 1**. A load cell is mounted at midspan of each $\frac{1}{2}$ curve to continuously measure the load required to deform the piece as it travels through the machine.

Using the average voltage readings of two load cells, the flat-wise bending modulus of elasticity of the timber can be estimated. The readings are continuously monitored as the piece travels through the machine yielding an estimate of the flat-wise modulus of elasticity profile along the length of the piece. The average and minimum flat-wise modulus of elasticity values are used as grading criteria to preliminarily sort the material into various machine stress grades. In most countries, human graders further inspect each piece after the machine grading for possible downgrade based on the presence of visual defects including wane, machine-induced defects, excessively large edge knots, etc. Special attention is paid towards both ends of each piece as the grading machine is unable to obtain stiffness estimate for these region. Recent developments include the use of non-contact X-ray grading machines where the principle is based on the positive correlation between the density and stiffness and strength of wood.

In Australia and New Zealand radiata pine is commonly machine graded. In North America, Douglas fir and spruce–pine–fir are also graded mechanically. In Europe, machine grading is available with the European spruce and pine species.

Machine-controlled versus output control systems

The setting of grade boundaries of the grading machine is critical in a machine grading mill. There are two distinct philosophies dealing with the quality control and grade boundary setting issues. In North America, an 'output control' system is adopted where the grade boundary settings of each machine are individually set based on a yield study of the relationship between strength and stiffness of the relevant timber resource. A quality control procedure known as the CUSUM method is employed to monitor and control the edgewise bending stiffness and strength of each grade where five pieces per grade are randomly selected for testing from the production of every 4-hour shift. Slight adjustments to the machine grade boundary control setting are allowed to account for variability of the resource. Excessive adjustments act as a trigger that necessitates cumbersome retesting procedures which tend to be avoided by the operators. Grading of spruce–pine–fir species tends to be stiffness controlled, i.e. when the modulus of elasticity is under control, the strength properties will also be regarded as 'controlled'. Douglas fir, as a large knotted species, tends to be bending strength controlled; the grade boundary settings tend be set to produce a product with slightly higher modulus of elasticity to keep the strength in check.

In Europe, the machine control system is adopted where machine settings are established by control authorities and cannot be adjusted. There machine settings tend to be conservatively set. Also, continuous quality control monitoring is not required with this system.

Stress classes versus species- and grade-specific approach

The strength properties of sawn timber are species and grade dependent. In some countries (e.g. the US and Canada), design codes for timber structures provide species- and grade-specific characteristic design properties for wood members. In other parts of the world, such as Europe a strength class system is used that assigns structural timber to grades or strength classes with defined properties. Using the North American system, the designer selects the species and grade during the design process; however, problems can arise if the designer is unfamiliar with the available species and grades of sawn timber in the region where the structure will be built. It is possible that the species and/or grades that are listed in design codes are not produced commercially on a regular basis; therefore, the cost of construction will increase significantly if products are transported long distances, or the designer/builder may have to redesign the structure using an alternative species or grade combination. The advantage of the species/grade-specific approach is that the set of strength properties associated a particular species and grade combination can closely reflect the 'true' properties of the timber.

Some designers support the idea that engineering design of timber structures could be simplified by introducing a classification system wherein the individual species and grades are grouped into a broader system of strength (or

stress) classes, i.e. within a specific class all species/grades be assigned the same design properties. The strength class approach is species independent and the choice of a species/grade combination within a strength class would be selected based on availability and price. The European CEN standard EN 338 provides strength classes for use in the European Union. The disadvantage of the stress class approach is that some of the strength properties associated a particular species and grade combination may be slightly compromised to fit within a stress class.

Reliable test data are required to establish the characteristic properties for design regardless of which approach is to be considered. Since the 1970s, recognised testing agencies from different timber-producing regions in the world have developed large databases on the strength properties of full-size timber (in-grade testing) produced in an individual country or region. Regional standardised test methods, which are now being harmonised into ISO Standards, are followed to develop the in-grade test data and regional procedures are used for deriving design properties of timber from test data. For example, in Europe the strength properties are derived according to European standard EN 384:2004 – *Structural Timber – Determination of Characteristic Values of Mechanical Properties and Density* – and the characteristic properties are calculated according to European Standard EN 338:2003 *Structural Timber – Strength Classes*. In the US and Canada, sampling and testing of sawn lumber is conducted according to ASTM D 4761:2007 *Standard Test Methods for Mechanical Properties of Lumber and Wood-base Structural Material*. In the US, the characteristic design properties are derived according to ASTM D 1990:2007 *Standard Practices for Establishing Allowable Properties for Visually-graded Dimension Lumber from In-grade Tests of Full-size Specimens*. In Canada, the characteristic design properties for sawn lumber are derived using procedures developed by Foschi *et al*. (1989). In Australia, the methods of test and procedures for deriving characteristic properties are defined in Australian and New Zealand Standards AS/NZS 4603:1992 *Timber – Stress Graded – In-grade Strength and Stiffness Evaluation*. Harmonisation procedures are needed to allow easily comparisons of data from different regions, especially if an international stress class system were to be developed.

Engineered wood products

Glue-laminated timber

Glued-laminated timber (glulam) is a structural timber product made of elements glued together from smaller pieces of wood. Glulam has a wide range of applications including beams and headers in residential buildings to girders, columns, and truss members in non-residential buildings. As glulam can be produced as long straight beams or complex curved arches, designers can express their artistic concepts in a building while satisfying the strength requirements with glulam products. Glulam structural forms are usually exposed within a building as an architectural feature to take advantage of their warm and pleasing appearance. **Figure 2** shows some examples of structural applications of glued-laminated timber. Foschi and Barrett (1980), Falk and Colling (1995), Serrano and Larsen (1999), Serrano (2002), and Lam and Mohadevan (2007) provide useful reviews on the performance of glulam members.

Glulam manufacturing uses a special grade of timber, known as laminating stock (lamstock). The lamstock is end jointed using finger joints to form the long laminates and they are further glued together in layers or laminations to form the structural member. This manufacturing process allows higher-grade lamstock to be used in high-stress zones, thereby maximising the strength-to-cost ratio.

Glued-laminated timber is manufactured according to production standards at certified plants. These standards govern the process of qualification/certification of the plants as well as the day-to-day production procedures of grading the timber, end joining the lamstock, gluing procedures, finishing techniques and quality control process.

Canadian glulam manufacturing is qualified under Canadian Standards Association Standard 0177 which provides guidelines for equipment, manufacturing and record-keeping procedures. In addition, Canadian glulam is manufactured in accordance with the requirements of Canadian Standards Association Standard 0122, Structural Glued-laminated Timber. Under these manufacturing procedures, representative glue bond and end joint samples are tested for strength adequacy. The fabricated glued-laminated timber members are also required to have a comprehensive quality assurance record on the lumber grading, glue bond and end-joint test results. In addition, information on the laminating conditions including glue spread rate, assembly time, curing conditions and curing time are recorded. Independent certification agencies perform scheduled mandatory audits to ensure that in-plant procedures meet the requirements of the manufacturing standard.

In the US, glued-laminated timber is manufactured in accordance with the requirements of the American Institute of Timber Construction AITC 103 Standard for Structural Glued-Laminated Timber (ANSI/AITC A190.1). In Japan and Europe, glued-laminated timber is manufactured in accordance with the requirements of Japan Agricultural Standards JAS 111 *Strength Class by Species and Lamina Properties*, and European Standards EN 386:2001 *Glued Laminated Timber – Performance Requirements and Minimum Production Requirements*, and EN 1194:1999 *Timber Structures – Glued Laminated Timber – Strength Classes and Determination of Characteristic Values*, respectively. Typically, the glued-laminated timber producers can

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Figure 2 Structural applications of glued-laminated timber

make available a certificate of product conformance upon request by their customers.

The most common species for glued-laminated timber production include Douglas fir, larch, spruce, pine, southern pine, yellow poplar, radiata pine and Norway spruce. Good-quality glue lines require the lamstock to be dried to a moisture content range of 7% to 15% and then machine finished (not sanded) to a uniformly smooth surface with a thickness tolerance of ± 0.4 mm. The lamstock is then visually graded and subjected to a non-destructive bending stiffness test to estimate its modulus of elasticity. The visual grade and stiffness assessments are then used as the basis for defining the grade of the lamstock piece and determining its placement within a glued-laminated timber member. In Europe, some facilities use X-ray based non-destructive test devices to grade the material.

The assessed material is then end-trimmed and profiled for end-jointing into full-length laminates using either finger-jointing or scarf-jointing techniques. Scarf joints are stronger than finger joints. Finger jointing is less wasteful but requires carefully detailed control of quality of the profiling of the fingers and gluing process. In a finger joint, it is desirable to have a combination of thin tip, flat slope and narrow pitch for high strength. Longer fingers will yield higher joint strength while shorter fingers offer more savings. Phenol-Resorcinol and melamine-based resins are examples of adhesives used in glued-laminated timber production. Other approved exterior structural adhesive can also be used. Typically the glued end joints are cured with heat using radio-frequency to produce the full-length laminates.

Glue is applied to the wide surface of the laminates which are then arranged in layers according to product specification. If a wide glulam member is required, multiple boards can be placed side by side with the edge joints staggered in the member. The assembly is placed under pressure using clamps (>0.7 MPa) for curing. Variables that affect curing include the curing time, temperature, and relative humidity in the curing location, temperature of the glue-line and other pertinent controls that adhere to the glue manufacturer's specifications. Sometimes heaters are also installed

in the curing location to maintain the temperature at the desired level.

After curing, the product is surface planed, patched, and end-trimmed. Some facilities also provide special drilling and notching for connections, and special sanding and staining for finish. The final glued-laminated timber product receives a coat of protective sealer before shipping and is normally wrapped for protection during shipping and erection.

Wood I-joists

The wood I-joist was successfully introduced into the construction market in the 1970s as an alternative to larger-dimension solid sawn timber to be used as joists and beams in both residential and commercial buildings. The production of wood I-joists in North America increased from 36 million linear metres in 1987 to over 360 million linear metres in 2006. A wood I-joist is made by gluing flanges to a web to form an I-shaped engineered wood product. The flanges are made with either solid sawn timber (either visually stress graded or machine stress rated lumber), laminated veneer lumber or laminated strand lumber. The web is made with either oriented strand board (OSB) or plywood panels. A review of the research related to the strength properties of wood I-joists is available from Forest Products Society (1990) and Foschi and Yao (1993).

The critical elements in wood I-joists are the flange-to-web connection and the joints between adjacent panels in the web. Individual manufacturers have proprietary flange-to-web joints that are protected by patents. In the web, adjacent panels are typically joined together using scarf jointing, or tongue-and-groove jointing techniques. Approved exterior structural adhesive such as Phenolic resin or polymeric Isocyanate (diphenylmethane di-isocyanate MDI) are used for the web-to-web and web-to-flange joints. Long-length wood I-joists can also be manufactured by finger-jointing shorter members together to form the flanges. Example applications of wood I-joists are shown in **Figure 3**.

Details of the manufacturing process for wood I-joists vary among different producers. Flange stock with a moisture content range of 8–18% is normally used. Finger-joining techniques are used to end-join shorter members to form longer flange members. A central groove is routed into the flange member to house the web material. Sufficient contact area between the web and the flange is needed to transfer the longitudinal shear stresses via the adhesive when the member is under load and different products have unique geometrical parameters for this groove.

The OSB or plywood web stock has a moisture content of approximate 5 to 10%. The ends of the web members of some products are profiled for the web-to-web connection; however, butt connections between adjacent web members are most often used. The edges of the web member are profiled to mate with the flange. Again these profiles are designed to increase the contact area required for the transfer of the longitudinal shear stresses via the adhesive between the flange and the web. The web-to-web and web-to-flange connections are formed under pressure using an approved exterior (durable) structural adhesive. The members are then cured at temperatures ranging between 21°C and 65°C. Final product inspection is performed and the product is wrapped for weather protection during shipment.

ASTM Standard D5055 outlines procedures for establishing strength values and monitoring the structural capacities of prefabricated wood I-joists. By following these procedures and meeting the acceptance criteria of individual agencies, manufacturers of wood I-joists can obtain product evaluations on their wood I-joist products. Ongoing in-house quality control programmes and independent third-party quality audits conducted by an accredited certifying agency on a regular basis are also necessary to ensure product quality.

Structural composite lumber

Structural composite lumber (SCL) is a generic term used to describe a family of engineered wood products that combine wood veneer or strands with exterior structural adhesives to form timber-like structural members. Here the wood veneer or strands are typically aligned with the grain angle of the strands or face grain of the veneers principally oriented along the length of the member. In some cases, members can also be made with orthogonally arranged layers of fibres. The manufacturing processes for structural composite lumber eliminate or disperse large defects such as knots and other strength-reducing characteristics throughout the cross-section to produce a more homogeneous and less variable product. Examples of structural composite lumber include: laminated veneer lumber (LVL) or Kerto which uses veneer sheets; parallel strand lumber (PSL) which uses veneer strands; and laminated strand lumber (LSL) and oriented strand lumber (OSL) which use flakes.

Parallel strand lumber was invented in Canada and launched as a commercial proprietary product Parallam® PSL by MacMillan Bloedel Limited in 1986. Currently the product is now produced by Trus Joist™ – a Weyerhaeuser business. This product is manufactured by bonding veneer strands with a phenolic-based adhesive under heat and pressure. Currently, parallel strand lumber products are manufactured in facilities in both Canada and the US (Barnes, 1988a; Churchland, 1988). Few journal publications are available on the mechanical properties of parallel strand lumber. Some studies on this type of strand-based product are available for reference (Bledsoe, 1990; Bledsoe

Figure 3 Applications of wood I-joists

and Craig, 1990; Triche and Hunt, 1993; Wang and Lam, 1998; Clouston and Lam, 2001).

In Canada, Douglas fir veneers and in the US southern pine and yellow poplar veneers are used in parallel strand lumber production. Veneer sheets, 3.2 mm × 1.2 m × 2.4 m in dimension, are dried to a target moisture content and then clipped into strands of approximately 19 mm in width. The length of these strands can range from short flakes or strands up to 2.0 m long. It is desirable to remove short strands in order to keep the strand oriented as much as possible along the longitudinal axis of the member. A defect removal process is therefore introduced to remove the short and/or weak fibres. Within the strand transportation line, a gap is introduced in the conveyer belt system such that short/weak strands which cannot bridge this gap are dropped into a waste collection bin below the conveyers. The target is to include strands longer than 305 mm in the final product.

The strands are then coated with phenol-formaldehyde resins using either a roll spreader or dipping the strands in an adhesive bin. Dipping the strand will create complete coating but the excess adhesive needs to be eliminated via an air blowing system. The strands are then further transported to an assembly station where a continuous loose billet of resinated strands is formed. From the transportation and strand deposition system, the strands tend to align themselves along the length of the billet. Throughout the strand transportation process, mass flow systems are used to allow the operators to monitor and carefully control the amount of fibres to be included in the formation of the loose billet.

A belt press is used to consolidate the loose billet under pressure while the billet is heated to the curing temperature of the adhesive using microwave energy. Finally, a finished continuous billet of 280 mm × up to 482 mm in cross-section can be produced. Because of the continuous press process, the billet length is limited only by transportation constraints. The finished billet is then cross-cut to desired lengths, rip sawn to produce rough dimensions or custom sizes, and sanded down to finished dimensions. Edge gluing techniques can be used to form large-dimension members if required. Individual beam weighing is also performed as a final step to monitor and control the shipped density.

Examples of applications of parallel strand lumber are shown in **Figure 4**. Finally, it is worth noting that the voids in the cross-section of parallel strand lumber allow the member to accept preservative treatment with a very high degree of penetration depth. Therefore, when chemical protection against decay is needed, treated parallel strand lumber can be considered as a suitable option to meet durability requirements.

Laminated veneer lumber is made by gluing layers of wood veneer sheets together using exterior structural adhesive by processes that are basically the same as those used for production of plywood. Laminated veneer lumber is used to produce beams, headers (or lintels), flange stock for wood I-joists, and scaffolding planks to take advantage of its attributes including high strength, dimension stability, and uniformity. The grain orientation of each veneer layer is aligned parallel to the long direction of the member to achieve strong directional strength properties along the member length. In some special laminated veneer products, veneers are introduced into the lay-up in the direction perpendicular to the long axis of the member to enhance the strength properties orthogonal to the member length. This special arrangement is intended to particularly improve connection properties of the members (Cha and Pearson, 1994).

Depending on the location of the production facility, laminated veneer lumber is made with veneer from single species such as Douglas fir, larch, southern pine, yellow

Figure 4 Example of parallel strand lumber application

poplar, western hemlock, lodgepole pine, Nordic pine, radiata pine and spruce. In some products, mix species can be introduced in the member lay-up. The initial production step of laminated veneer lumber is similar to that of plywood where logs are rotary peeled on a lathe to yield 2.6 m wide veneer sheets. The veneer, which is between 2.5 mm and 4.8 mm thick, is cut into sheets 686 mm or 1372 mm in width and 2565 mm in length – noting that these dimensions vary slightly depending upon the location of production. After the veneers are dried to target moisture content, major strength reduction defects can be removed by a clipping device. Visual grading and/or non-destructive testing devices, such as ultrasound systems and density verification equipment with visual overrides, are used to grade the veneers.

The graded veneer sheets are then cut to the desired width and structural exterior adhesive such as phenol-formaldehyde resins are applied on the veneer. The resinated veneer sheets are then built up to form a long loose billet. The veneer sheets are typically end-jointed with lap or butt joints. Typically, veneers of high quality/grade are placed at the outer faces of a member during manufacturing. This is especially important if the member is used on the flat for scaffolding planks application where higher-quality material on the member faces improves the strength efficiency of the member and provides a better wearing surface.

After forming, the billet is fed continuously through a series of rollers that apply heat and pressure to cure the adhesive and consolidate the billet. **Figure 5** shows an example of common structural applications of laminated veneer lumber.

Laminated strand lumber and oriented strand lumber are structural composites with wood strands primarily oriented parallel to the long axis of the member. The least dimension of the strands is less than 2.54 mm. The average length of the strand in laminated strand lumber and oriented strand lumber is a minimum of 150 and 75 times of the least dimension, respectively. The first commercial laminate strand lumber product was launched in Canada by MacMillan Bloedel Limited in the early 1990s under the trade name TimberStrand® LSL. As a result of changes in company ownership, the product is now produced by Trus Joist™ a Weyerhaeuser Business.

These products aim to replace some larger-dimension solid sawn timber products using underutilised commercial species (Barnes, 1988b, 2000a, 2000b). Strands from fast growing hardwood species such as aspen-poplar are bonded with isocyanate as diphenylmethane di-isocyanate (MDI) or other structural adhesive under heat and pressure to form the product.

Fresh-cut logs are soaked in hot water ponds to soften the wood prior to debarking and stranding to reduce the

Figure 5 Examples of laminated veneer lumber application

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Figure 6 Examples of oriented strand lumber in rimboard application

amount of generated fines and slivers. After conditioning, the logs are debarked and fed into a stranding machine to obtain the desired strands. The strands are screened to remove fines and unwanted particles. A cylindrical dryer is then used to dry the strands to a moisture content range of 3–7%. It is desired to minimise breakage and maintain uniform moisture distribution of the strands during drying. After drying the strands are resin-ated in a blender with resins and wax. Resin content is proprietary information for these products. A careful monitoring process is instituted to regulate the relative weights of the strands and the resin. A former/orienter machine is used to arrange and deposit the strands into a loose mat with the strands primarily oriented parallel to the long axis of the mat and the weight of the loose mat is also monitored to check for uniformity of distribution of the strands.

The loose mats are then pressed under heat and pressure for the curing of the adhesives to form solid panels. The footprint of the loose mat depends on the mill. The idea is to heat the core through the surface of the mat as quickly as possible. Steam injection pressing, stream preheating processing or microwave-based technology can all be used to enhance the heating of the core material such that relatively thick members can be formed with a uniform density profile through the thickness of the member. These strategies can shorten the required press time by enhancing the heating of the core, which leads to cost savings from reduced press time. Non-destructive testing devices are used to scan the panels for defects such as blows. The finished panels are cross-cut and rip-sawn to produce rough dimensions or custom sizes, and then sanded down to finished dimensions.

Laminated strand lumber and oriented strand lumber area produced at a moisture content range of 6–8%. These products are strong and consistent and are less prone than sawn timber products to shrinking, warping, and cupping, bowing or splitting resulting from seasoning stresses. In residential wood frame construction, laminated strand lumber and oriented strand lumber are well suited for use as headers, lintels, studs and rimboard. Examples of oriented strand lumber in rimboard application are shown in **Figure 6**.

Conclusion

The demand for timber products as a green and renewable natural building material will keep increasing in the future. This chapter has provided a brief discussion on some of the most common timber products, their manufacturing process and the scope of their application. Since wood is inherently a versatile material, it can be easily reconstituted to form engineered wood products. These products utilise non-traditional resources and possess improved consistency of physical and mechanical properties when compared to traditional structural products.

In central Europe, a thick massive wood system using a cross-laminated timber technology has recently been introduced. These systems, with advantages of high thermal efficiency, good fire resistance and high strength, have led to opportunities to construct midrise timber buildings. Increased market opportunities have also led to newer engineered wood products using plant-based fibre resources such as bamboo and straw (Youngquist *et al.*, 1994; Rowell and Young, 1997). Hybrid structures that are made by combining wood with plastic or by reinforcing wood members with high-performance fibres such as glass, carbon, and Kevlar fibres are available (Youngquist, 1995).

Research and development efforts are still needed to fully understand the interacting relationship between the raw material, the manufacturing and processing variables, the engineering and physical properties of the product, and the end-use applications. Research on the application of

engineered wood products with respect to their connection details in heavy timber construction is another important topic. The performance and behaviour of engineered wood product systems under both ultimate and serviceability conditions must also be studied to expand their applications.

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Chapter 62

Effects of moisture

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Shrinkage and swelling of wood and wood products may be quantified according to changes in moisture content with humidity and temperature. Moisture-related movement in construction can be managed in design by allowing for movement and controlling the building process.

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Introduction

Wood in living trees contains moisture. Cells in the wood transport moisture in various ways to assist the growth of the tree and as a result moisture contents in trees are usually quite high. After any wood is cut from a tree, it begins to dry and interact with moisture in the environment. Variations in humidity and air temperature cause fluctuations in the moisture content of wood. These changes in moisture content affect the properties and dimensions of wood and wood products.

More problems in timber construction are attributable to inadequate detailing for moisture-related issues than anything to do with structural design. Designers must therefore consider how moisture affects the performance of timber components and structures, during manufacture, construction and in long-term service.

To avoid problems and build durable timber buildings, moisture content in wood should be managed at all stages of processing and use. This is essential to minimise shrinkage, swelling, distortion or splitting of wood and for the quality of processes such as gluing, machining and surface coating. Fungal and insect attack and metal corrosion are also affected by the amount of moisture in wood. To reduce the effect of changes in moisture, wood should be:

- Dried to the correct moisture content
- Stored properly to avoid undue moisture change before and during manufacture
- Manufactured under controlled conditions
- Protected properly before use and while being put into use
- Protected as necessary in the finished situation.

Any structure or component manufactured from wood-based products needs to incorporate appropriate allowances for moisture content control in the design, specification and detailing in order to ensure satisfactory long-term performance in service.

Water and wood

Wood in normal use always contains water. The water is either 'bound', i.e. combined with the wood cell walls, or it may be 'free' within the cell cavities. When wet wood dries, the free water is lost first. This causes little change in wood properties other than reducing weight. When drying is continued, until all the free water is lost and the cell cavities are empty, further moisture loss can only come from bound water. The loss of this bound water is accompanied by shrinkage and changes to mechanical properties such as strength and modulus of elasticity. The moisture content, at which there is no more free water and only water bound on the cell walls, is called the fibre saturation point or FSP and significant property changes occur when the moisture content drops below this point.

Moisture content

Moisture content is a measure of the amount of water in a piece of wood and is defined as:

$$\text{Moisture content (\%)} = \text{weight of water/weight of oven dry wood} \times 100 \quad (1)$$

This can be measured either indirectly with hand-held moisture meters or directly and most accurately by weighed samples oven dried until all water is driven off. (This can be confirmed by successive weighings.)

$$\text{Moisture content (\%)} = \frac{\text{weight when cut} - \text{weight oven dry}}{\text{weight oven dry}} \times 100 \quad (2)$$

Several types of moisture meter are available. The most common is the electrical resistance type which is reasonably accurate, measuring moisture content to within 1.5%. Those with short probes are easier to use but measure only moisture content near the surface, whereas those with longer probes can measure the core of timber. Some are surface contact only. Radio frequency types are only accurate to about 3% but are very useful as a rapid indicator. Moisture meter readings need to be corrected for species, treatment and temperature effects and meters should be correctly calibrated.

Example: Using the oven drying method to obtain wood moisture content. Assume a sample of wood weighs 5 kg. It is placed in an oven at 105°C until there is no loss in weight.

It now weighs only 4 kg, 1 kg of water having been driven off. The original moisture content was:

$$\text{Moisture content} = (5 - 4)/4 \times 100 = 25\%$$

Green (unseasoned) wood

Wood from newly felled trees is usually called 'green' wood and has high moisture content. The cell cavities contain water and the cell structure is swollen. The moisture content of green wood is often around 100%, but varies considerably between species, ranging from 45% to 200%. Low-density species such as balsa and redwood have a low mass of wood in a given timber volume and can have moisture contents exceeding 200%. Heartwood has lower moisture content than sapwood.

Dry (seasoned) wood

The term 'dry' wood must be used carefully, because all wood in service contains some moisture. A specification referring to dry wood must be qualified with a specified range of moisture contents to ensure that allowances for any change in properties or dimensions are of the right magnitude.

Equilibrium moisture content

Timber in constant conditions of humidity and temperature will gain or lose moisture until it comes into equilibrium with the atmosphere. It is then said to have reached its equilibrium moisture content (EMC) for those conditions. Because EMC varies with chemical composition, different species and wood products will reach different moisture contents under similar conditions. To minimise the effects of moisture content on wood properties, it is essential that wood be dried to (and maintained at) a moisture content as close as possible to the EMC of the final conditions of use.

Timber in a low-temperature, high-humidity environment has high EMC, whereas timber in high-temperature, low-humidity conditions has a low EMC. The general relationship for solid wood is given in **Figure 1**, from which the EMC may be calculated for any environment of known humidity and temperature. For example, a component destined for an environment with 40°C temperature and humidity 40% may need to be below 8% moisture content.

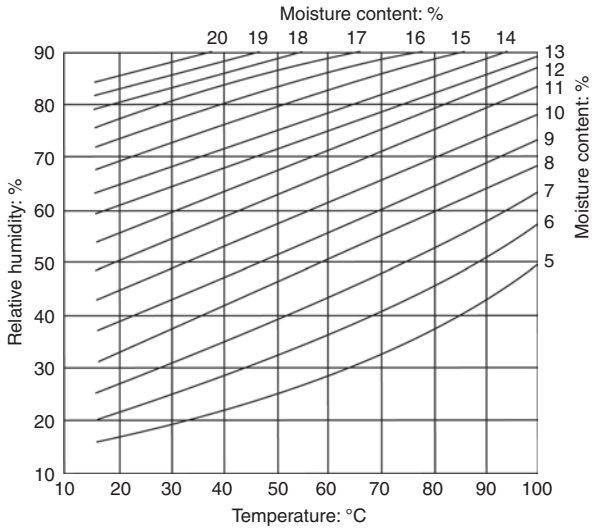

Figure 1 Equilibrium moisture content as influenced by temperature and relative humidity (Source: Forest Products Society)

Different countries have measured equilibrium moisture contents in typical environments and the recommended practice is to dry and supply wood that is as close to these as possible. Specific tables published for Britain (Building Research Establishment, 1986), USA (Forest Products Society, 1999) and New Zealand (Orman, 1966) show wide ranges across warm dry to temperate damp environments. Cold environments can also have very low EMC. In the USA, EMC ranges from 4% to 18% depending on the time of the year and on the locality. Table 1 gives the approximate EMC and therefore the recommended moisture contents for timber in various environments.

Designers and producers need to assess the target environment for their product and whilst they can be guided by **Table 1**, it is advisable to seek more detail from authorities in their jurisdiction (see references).

In other environments, some exterior components such as bridge beams attain moisture contents as high as 20% to 24% and piles beneath the soil water table and in

Use category	Air-conditioned buildings in Australia and New Zealand	Most of USA	Dry South West USA	Damp coastal USA	United Kingdom
Exterior finishing and items outside the insulation	14–18	9–14	7–12	9–14	13–19
Flooring exposed to ground atmosphere	10–14				
Interior joinery, and finish, wall framing, flooring not exposed to ground atmosphere	8–12	6–10	4–9	8–13	10–12

Table 1 Suggested moisture content for wood components in a range of international environments

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marine structures may be considered as saturated or 'green'.

Surface coatings reduce the rate of moisture movement into and out of wood and wood products, but they cannot entirely halt the process. Given sufficient time, wood ultimately comes into equilibrium with the average atmosphere of its environment.

Getting wood to the optimum moisture content is normally only possible using kiln drying.

Drying

Wood needs to be dried to achieve the correct moisture content for given service conditions. Methods include:

- air drying where timber is filleted (with spacers between layers) stacked and allowed to dry with the passage of natural air currents; and
- kiln drying using heat and forced air flow in a controlled cycle in a kiln.

These processes involve time and handling that can sometimes affect the delivery of timber orders. Air drying is rarely used except for high-value decorative timbers that are difficult to dry because it can take several weeks or months depending on the time of the year. Air drying results in higher moisture contents close to the external EMC indicted in **Table 1**. Lower interior moisture contents can usually be achieved with conventional kilns in a week depending on the species. For high-permeability species, special high-temperature kilns are increasingly used for drying in 24 h (Williams and Kininmonth, 1984). Wood in kilns is often stacked with weights to keep the boards straight and prevent distortion during the drying process. Thinner wood takes less time to dry and veneer used for plywood and laminated veneer lumber (LVL) manufacture can be dried to 5% moisture content in less than 10 min.

Shrinkage and swelling

Movement of wood

As the moisture content of wood reduces during drying, shrinkage commences at the point when the cell walls start to lose bound water. This is called the fibre saturation point (FSP) and is notionally about 25% to 30% moisture content, as listed in **Table 2**. Movement of green wood due to moisture changes above the fibre saturation point is negligible; however, for moisture changes below the fibre saturation point, wood will swell as it gains moisture and shrink as it loses moisture as shown in **Figure 2**.

Shrinkage of timber is usually determined as the change in dimension from green (fibre saturation point) to a specific dry moisture content, divided by the green dimension, expressed as a percentage.

Example: If a piece of wood measures 150 mm wide in the green state, and 144 mm when dried to 12% moisture content, its width shrinkage is:

$$(150 - 144)/150 \times 100 = 4\%$$

Species	% shrinkage, when drying from green to 12% m.c.		Fibre saturation point (%m.c.)
	Tangential	Radial	
Radiata pine	3.9	2.1	29
Douglas fir NZ	4.9	2.8	27
Douglas fir USA	4.0	2.5	
Redwood	2.2	1.3	25
Western Red Cedar	2.2	1.1	
Eucalyptus spp.	11.0	6.4	30
Oak	5.0	2.6	

m.c., moisture content

Table 2 Approximate shrinkage properties of some timbers

The amount of movement is proportional to the change in moisture content and varies with species, density and the direction of the grain. Dense woods generally shrink and swell more than lighter woods. Some authorities (Standards Austrialia; Buchanan, 2007) express the specified value as 12% (close to in-service EMC) but others (Forest Products Society) use the dimension at oven dry. Values in **Table 2** assume that the timber is green above FSP and some are estimated from oven dry numbers using a ratio of 12% to an assumed FSP of 30%.

It is clear from Table 2 that radial movement is about half the tangential value, and the range between species can be quite large. There is also significant variation between individual pieces with coefficients of variation of more than 15%. Designers should refer to national standards or texts

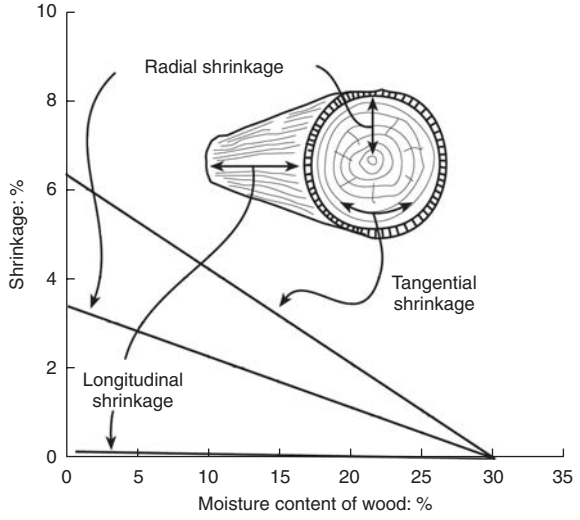

Figure 2 Shrinkage at various directions to the grain

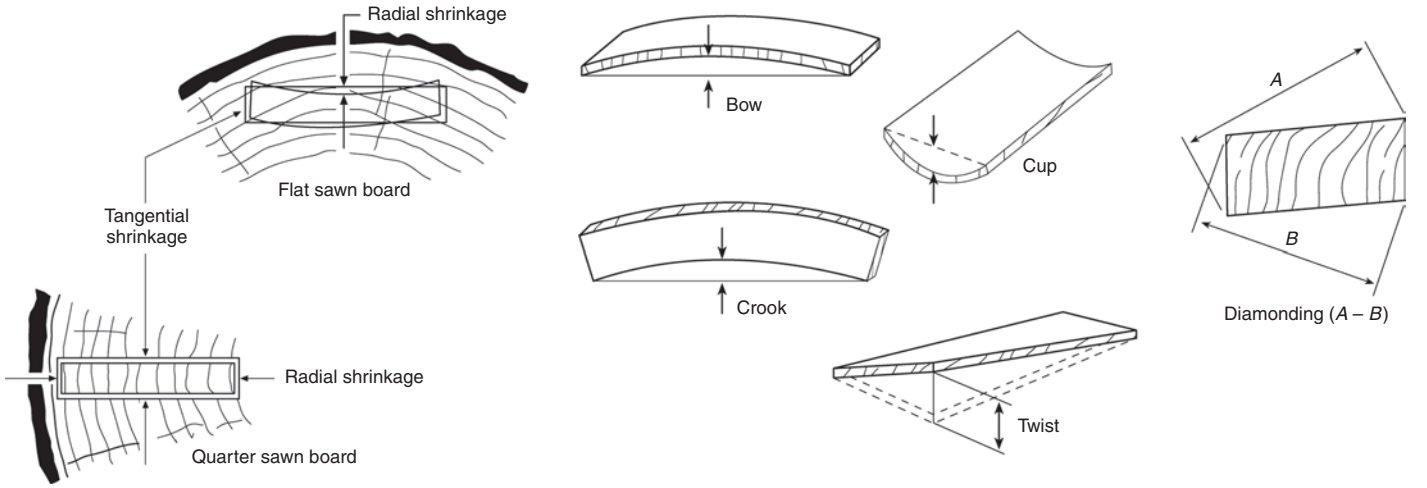

Figure 3 Distortion of boards on drying, caused by differential shrinkage in different directions

on wood properties. For example, AS/NZS1720.2 Part 2 Timber Properties (Standards Australia, 1997a) tables tangential shrinkage values for species commercially available in Australia, and more detail is available in referenced publications (Bolza and Kloot, 1961; Forest Products Society, 1999).

Transverse shrinkage

The difference between tangential and radial shrinkage is the cause of much distortion and degrade in drying, such as:

- cupping of flatsawn boards as shown in **Figure 3**;
- diamonding of square sections as shown in **Figure 3**;
- checking and splitting of poles;
- checking and splitting of large beams which enclose the pith.

Limits are placed on distortion by timber grading rules.

Glue laminated timber and finger-jointed timber are both more stable than single long lengths of timber because they contain smaller pieces of wood and the varying movements of adjacent pieces are averaged.

Longitudinal shrinkage

Longitudinal shrinkage (along the grain) of most timbers is about 0.1% when drying from green to 12% moisture content (m.c.). This is less than one tenth of the transverse shrinkage. Thus, for normal timber in service, the small seasonal variation in moisture content of 2% to 5% will result in only small changes in length which are usually inconsequential. Longitudinal movement is only likely to be a problem where long members supporting critical elements are subject to differential movements producing visible deflections or undesirable stresses. In some species wood from the centre of the tree (near the pith) may shrink more than normal wood and the resulting uneven longitudinal shrinkage can sometimes cause severe distortion or warp (crook, bow, twist) if moisture content is not controlled (**Figure 3**).

Calculating moisture movement of wood

Transverse movement

Designers, specifiers and builders have little or no control of the orientation of the radial and tangential components of structural timber. It is therefore appropriate to use an average value when calculating shrinkage or swelling of structural elements. Movement can be variable so accuracy to within less than a millimetre is probably not warranted. Using the shrinkage values in **Table 2** the change of a given cross-sectional dimension can be calculated from:

$$\text{Dimensional change} = \%\ \text{shrinkage Table 2}/100 \times \text{m.c. change}/(\text{FSP} - 12) \times \text{initial dimension} \quad (3)$$

Figure 4 enables a rough estimate of long-term transverse moisture movement, based on the above calculations, using an average transverse shrinkage of 3% from **Table 2**. It shows a simple rule that for every 12% change in moisture content a 100 mm board will move 2 mm. This can be used to give a simple formula:

$$\text{Dimensional change} = 0.0017 \times \%\ \text{m.c. change} \times \text{initial dimension} \quad (4)$$

where the 0.0017 is an average dimensional change coefficient for radiata pine. USDA *Wood Handbook* (Forest Products Society, 1999) tables coefficients for many species both radially and tangentially that can be used to calculate movement in typical service range (6% to 14%).

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Longitudinal movement

For almost all timbers, longitudinal movement can be estimated from:

$$\text{Change in length} = 0.00005 \times \text{initial length} \times \%\ \text{m.c. change}$$

This may usually be neglected in design.

Consider a 4.8 m long post of green timber, installed in a situation where the final EMC is 12%. The shortening of the column for a change in moisture content of $30 - 12\% = 18\%$ is $0.00005 \times 4800 \times 18 = 4.3$ mm. If adjacent supports are steel or masonry, there will be differential deflection of 4.3 mm. Pith or corewood in the column could lead to larger movement. Simple precautions would be to specify that the column should be pith free, dried to 12% before installation, and protected from rewetting during construction. For a critical member such as this, glulam or LVL would be more appropriate, manufactured dry and protected during construction.

Distortion

Distortion of wood and wood products occurs in many different ways, mostly due to differential shrinkage during drying. **Figure 3** shows the most common forms of distortion occurring in sawn timber.

Movement of panel products

Wood-based panel products generally have smaller moisture-induced changes of expansion and contraction than solid wood, because of the influence of adhesives and manufacturing processes. **Table 3** gives typical values. Shrinkage as a percentage is very low for plywood because of restraint provided by the parallel-to-grain plies in each direction. However, sheet materials are much wider than sawn boards so appropriate allowance for movement is needed. For thickness swelling, some coefficients may be as much or more than for timber, but this movement is generally negligible for thin panels. There are wide variations between different panel types. Panels often swell faster at the edges where there is greater moisture absorption. Edge priming is recommended to control this differential swelling.

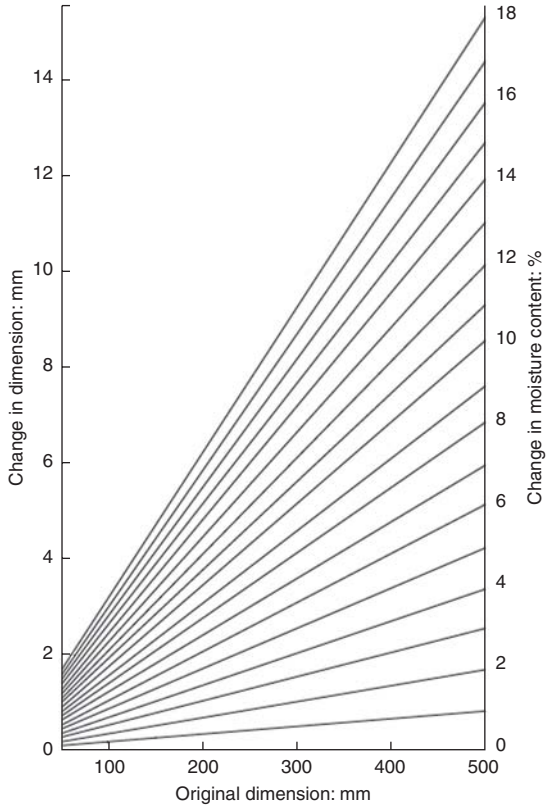

Figure 4 Typical changes in dimension due to change in moisture content. This chart is based on average tangential and radial shrinkage of 3% when drying from green to 12% moisture content

Calculating moisture movement of panel products

Plywood movement can be related to the properties of the wood from which it was made. Transverse movement across the grain in the plane of the veneer is constrained by minimal longitudinal movement along the grain in adjacent veneers crossing at right angles. Therefore, movement in the plane of the panel is only about double the longitudinal movement of timber. The change in the

Product	Likely moisture content ex factory	EMC: Very dry conditions (50% relative humidity, 20°C)	EMC: Very humid conditions (90% relative humidity, 20°C)	Linear expansion coefficient/ mm/%m.c.	Thickness swelling/ mm/%m.c.
Plywood	10–14	8	19	0.0001	0.001
Particle board	16–14	10	17	0.0003	0.002–0.004
MDF		7	12	0.0005	0.01
Hardboard		6	12	0.0003	

Table 3 Moisture content and movement properties of some panel products

width or length of a sheet is therefore about 0.0001 × initial dimension × % m.c. change. Movement in the thickness is almost unrestricted and dimensional calculations for plywood thickness are similar to those for solid wood. Swelling of plywood is recoverable on drying.

For most other panel products the fibre directions are random, so the change in dimension can be obtained by multiplying the initial dimension × % m.c. change × coefficient from **Table 3**. Depending on the adhesive used, particle boards and fibre-based panels will swell permanently in their thickness and never recover the quite large movements that may occur due to loosening of the fibres.

Movement of large timber components

With large timber components, even a small change in moisture content can result in significant movement that might affect detailing and connection requirements, which in turn can induce significant stress concentrations that were not anticipated during design. On the other hand, changes in moisture content will be very slow in large timber members. The use of laminated timber or LVL has advantages in the control of moisture movement because:

- manufacturing is at a moisture content close to EMC (gluing requires that timber be dry);
- there is increased stability from the random gluing together of pieces;
- glulam and LVL are handled and stored carefully because of their high value;
- these products can be wrapped or coated with water repellents before delivery.

Large solid sawn timber beams such as 300 × 100 mm may split on one face if they contain the pith of the tree. There is no way of preventing this. If appearance is important the beams should be selected carefully from previously dried stock, or they should be cut from the outer part of the log so that they are free of pith.

Water, wood and durability

Wood-destroying organisms thrive in damp wood close to or a little wetter than the fibre saturation point at comfortable ambient temperatures around 20 to 25°C. In these conditions oxygen is normally available for fungi and insects, food is present in the cell walls or wood sap, and if there is a source of infection the wood cell walls may decay and/or be consumed by termite or borer insects.

Dry wood kept dry will not generally decay, so the rules that apply to control of wood movement will assist in controlling decay. Thoroughly saturated wood will not decay either because of the lack of oxygen, but there may be bacterial degradation. There have been numerous instances where valuable decorative timbers have been recovered after centuries of submersion from swamps and for reuse as ornaments and the like. Untreated timber pile foundations below the water table can also remain durable as long as there is no risk of the moisture content dropping below full saturation.

In most buildings, the key to durability is to keep moisture content below about 20% so that fungi do not take hold. The worst-case scenario occurs when a water transporting fungus (the so-called 'dry rot' fungi) takes hold while wood is wetter, and this grows and extends to a moisture source in nearby soil from which it can bring its own water to the food source.

To avoid decay problems, the wood should be kept dry by careful detailing during both design and construction. Design details should provide barriers (damp-proof membranes) between wood and concrete or masonry, and allow for good ventilation of components. If there is a risk of elevated moisture content, the wood can be chemically impregnated (preservative treated) or thermally modified to make it less palatable to fungi. Likewise, insect attack can be prevented with preservative treatments. Some insects (termites) will attack dry wood but apart from treatment it is possible to exclude the target insect by design using appropriate detailing. A unique Australian standard AS 3660 covers these methods (Standards Australia, 1997b). However, chemical treatment is not an excuse for bad design and many parts of Europe have developed a strong tradition of good design detailing without resorting to the use of preservative chemicals – many of which are now banned from use in some countries.

For wood that is exposed to the elements, durability is reduced by the cycles of shrinking and swelling that occur as described above. These cycles stress the bond between surface and inner fibres, and splitting or cracking can result. Unprotected surfaces are also oxidised and degraded by sun radiation. Coatings and detailing can minimise these problems. Coatings reduce the splitting stresses on the surface by slowing the transfer of liquid rainwater, and good colour choice can cool the surface. Coatings reduce radiation and oxidation effects, but a dark coating can actually increase problems by raising surface temperatures and inducing stress.

Construction practices

Sources of moisture

Most movement of wood in service results from changes in moisture content, so it is important for designers to ask 'Where will moisture changes come from and what can I do about them?' Typical sources of moisture change that need consideration in design, detailing and protection or control are:

- drying from green to the equilibrium moisture content;
- wetting due to rain, during transport or construction;

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- seasonal movement from dry summer to damp winter, or winter to summer;
- drying due to central heating in winter;
- wetting from high-humidity environments (swimming pools or tropical sites);
- condensation from higher-humidity interiors cooling on cold exterior cladding or roofing (or vice versa in tropical construction);
- site moisture from drying of concrete or other materials;
- moisture from soil or ground atmosphere;
- leaks from plumbing fittings.

Keeping wood dry during construction

Where control of movement is important, timber should be installed 'dry' at the expected equilibrium moisture content and protected until the design conditions are established. Allowing dried timber to get wet is often a costly error, and subsequent air drying in place may not return the wood to its design moisture content. Timber performs best when it is installed dry, kept dry and used dry. The examples that follow show some problems that designers may need to anticipate.

Careful storage of timber is essential to prevent problems occurring between drying of the timber and final finishing. Both sun and rain can cause deterioration. Moisture uptake in timber exposed to the weather is extremely rapid for some species. For timber exposed to the weather, defilleting and block stacking may reduce moisture pickup to an acceptable level, especially if the timber is wrapped in waterproof paper. However, it should also be recognised that moisture can get trapped in a parcel of timber if the wrapping membrane is damaged.

Great care must be exercised to ensure that large dry timber members such as glulam or LVL do not become wet during construction. No fully impervious coatings exist, but temporary protection from rain wetting can be provided by application of coatings or proprietary sealants. However, to be effective these must be applied in the correct way and they must be maintained on the site.

Dry members installed in a structure should be sealed or remain wrapped until the roof is installed. Partial or damaged wrapping can expose timber to sunlight in patches, causing unsightly appearance, and it can entrap moisture. All timber and timber products on site should be:

- stacked on well-aligned bearers;
- kept well clear of the ground or concrete floor slab;
- covered to protect from sun and rain;
- kept in original wrapping if delivered wrapped.

Light timber framing

Framing timbers should preferably be dried before delivery. If not, to minimise distortion, green framing timber should either be carefully fillet stacked for air drying on site or alternatively block stacked with no spaces between layers to prevent drying, and be installed in position in the structure before drying commences. If wall studs are installed in the green condition, much of the drying distortion can be resisted by nailing solid timber blocking between the studs.

If any framing timber is installed in the green condition, shrinkage will occur as the wood dries from the fibre saturation point to the equilibrium moisture content. This drying may take only a month in unlined buildings in summer, but if the timber frame is enclosed with a vapour barrier on one side and low permeability cladding on the other, drying and shrinkage may take two years before reaching equilibrium. In addition to shrinkage and distortion, the moisture released from green wood can cause condensation and other problems with lining and insulation materials. Accordingly, most buildings are built with kiln dry framing or with engineered wood products such as glulam and LVL which are manufactured dry and delivered dry to the building site.

The use of green framing may be appropriate in well-ventilated roof spaces, or light industrial or agricultural buildings with open ventilated structure. The use of dry framing is essential in flat or skillion roofs where impermeable linings enclose timber in a confined space.

Multi-storey framing

In multi-storey light timber frame buildings, the vertical movement at each floor is the sum of the shrinkage (perpendicular to the grain) in the top and bottom plates and the floor joists or lintels. Small movements in several floors can add up to large movements over the height of the building. Expansion and contraction gaps need to be provided in exterior claddings and interior linings, especially when they run full height such as in stairwells. Much of the movement can be reduced with the use of dry engineered wood I-beams instead of green timber joists.

Lintels and creep

Creep is a long-term deflection that occurs under constant load. In practice, creep is usually only a problem in the design of beams such as lintels where there is a need to limit deformation over fixtures such as sliding doors and windows. Problems attributed to creep may often be due to inadequate allowances for shrinkage. The two problems are often related because creep is greatly affected by changes in moisture content. Where deformation is critical, it is recommended that dry timber be used, or green timber beams should be propped while drying on site. Calculation of creep deflection is discussed elsewhere in this manual.

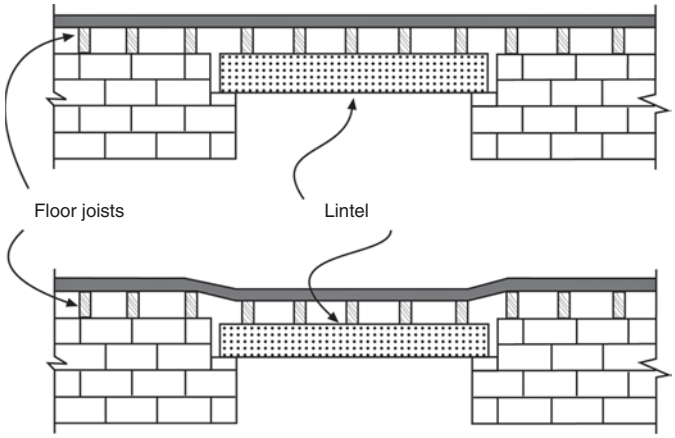

Figure 5 Deformation in floor due to shrinkage of timber lintel (300 mm deep)

Differential movement

Timber adjacent to stiffer elements

Most moisture movement problems in service occur when timber movement is compounded by its association with stiffer elements, or with fittings such as doors and windows. Consider the arrangement in **Figure 5**. The masonry blockwork does not move but, if the lintel is installed green, approximate shrinkage for 18% change in moisture content is $0.0017 \times 18 \times 300 = 9$ mm. Installing timber at the correct moisture content would eliminate this. Green timber joists framed into dry timber, steel or concrete beams should be kept proud to allow for shrinkage movement, as shown in **Figure 6**.

Alterations and additions

Vertical movement due to shrinkage of joists and plates in a new building such as a house is often of minimal concern if the whole structure moves. However, installing green joists and plates next to an existing structure of dry framing can contribute to significant differential movement at both floor and ceiling levels.

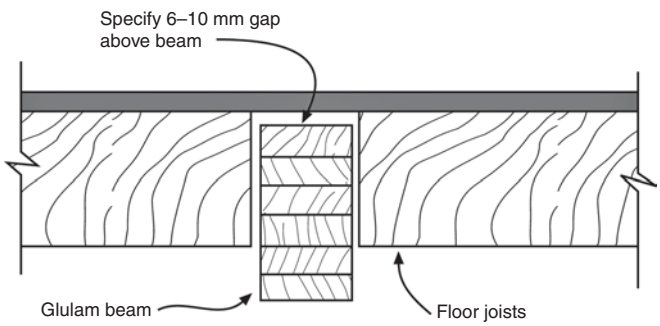

Figure 6 Gap above the glulam beam to allow for possible shrinkage in floor joists or swell in the dry beam

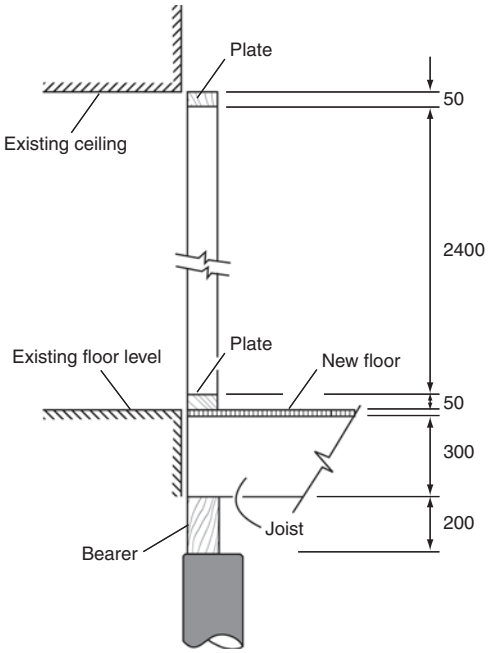

Figure 7 Section through a timber-framed addition to an existing house

Consider the addition of green framing such as 300×50 floor joists on a bearer and two 100×50 plates in the walls, as shown in **Figure 7**. From **Table 1**, the final EMC can be estimated as 14%. If the FSP is 30%, the differential movement at floor level would be $0.0017 \times (30 - 14) \times 500 = 13.6$ mm. At ceiling level there is additional shrinkage in two plates of $0.0017 \times (30 - 14) \times 100 = 2.7$. Total movement at ceiling level would thus be 16 mm. This should be allowed for in design, or timber should be pre-dried to the same moisture content as the existing building. Use of engineered I-beams can avoid this problem for two reasons: first they are supplied dry and, second, if they get wet the panel-type web material (often plywood) does not move as much as solid timber.

Rigid connections

When shrinkage occurs in timber that is rigidly fastened to steel side plates in a joint, the timber will be stressed perpendicular to the grain in tension, as shown in **Figure 8(a)**. This is a weak stress mode for timber and severe splitting can develop, particularly in bolted joints. The issue must also be considered in portal joints (**Figure 9**). Problems can be avoided if:

- the timber is installed at the correct moisture content;
- the rows of bolts are closely spaced, to minimise relative movement;
- slotted holes for one row allow transverse movement, as shown in Figure 8(b).

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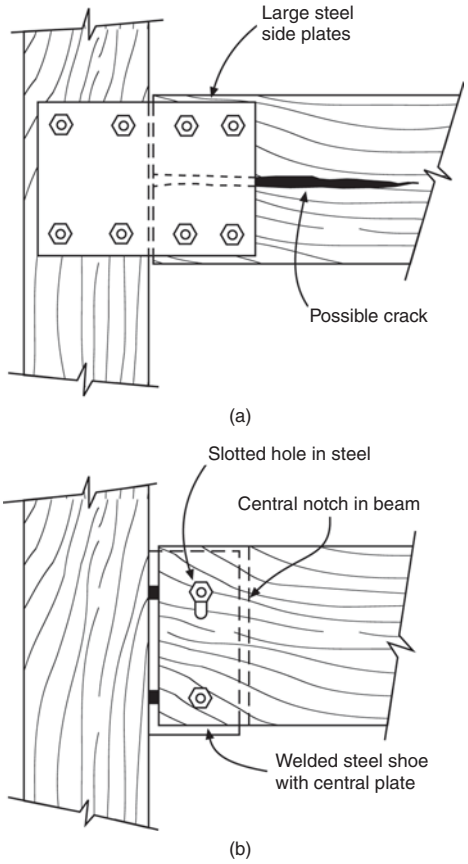

Figure 8 Alternative details for timber beam–column connection: (a) poor connection with beam restrained against shrinkage; (b) improved connection with bearing for beam and provision for shrinkage

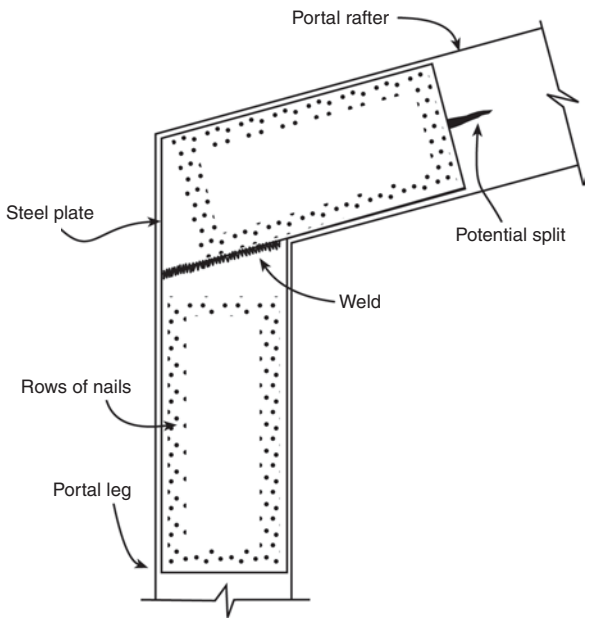

Figure 9 Example of restraint from a portal connection, resulting in splitting due to shrinkage

Movement of timber flooring

Wide expanses of timber flooring have the potential for large movements under changing moisture conditions. Several different flooring systems are considered below.

Sheet flooring

Most domestic floors are constructed from large panels of particle board or oriented strand board (OSB). The common practice of pre-laying and exposing floor panels to the weather allows the panels to shrink and swell as the moisture content changes. Manufacturers' literature sets out preferred procedures and expansion details for installing flooring panels. It is good practice to leave a loose strip for fixing on completion.

The edges of sheet flooring may be tongue and grooved. Properly designed edge profiles can be used to accommodate swelling. In some plywood flooring, the upper tongue is wider than the lower tongue, allowing the sheet to expand while the upper tongue compresses and recovers, so that sheets when installed may be loosely butted rather than having 2 mm gaps.

Laminated floors

In air-conditioned commercial buildings, the final moisture content can be much lower than the moisture content at manufacture. Problems can be avoided by using specially dried timber, protecting from the weather and providing details which allow some movement. The floors should be laid after the roof is in place.

Example: A floor made from 300 mm wide laminated panels is manufactured and installed at about 14% moisture content. The building is to be air-conditioned and the expected EMC is 8% from **Table 1**. This gives a moisture content change of 6%. From **Figure 4**, shrinkage in each panel can be expected to be 3 mm (causing gaps between panels).

Conversely, the same floor might be subjected to sustained rain wetting during construction. If the moisture content increased by 6% and the total width of the floor was 9 m (30 panels), the overall increase in width would be $3\,\text{mm} \times 30 = 90\,\text{mm}$. This could impose severe stresses on fasteners or secondary elements, especially if the floor is restrained between masonry or concrete walls. A suitable expansion strip would need to be provided (say at the edge of the floor), and it could be filled in on completion, with delay of final fixing of the panels. However, to avoid both shrinkage and swelling problems, timber should be kiln dried to 8% or 10% m.c. and be protected during construction.

Strip flooring

Strip flooring is purposefully narrow to minimise the shrinkage gaps between boards. It must be ordered dry

and kept dry before fixing. Joists should be less than 18% moisture content before fixing the floor to minimise gaps and squeaks in the floor. A bead of construction adhesive between the flooring and the joists will help to prevent squeaking.

In strip flooring, tight fitting of the tongue-and-grooved edges is essential for control of cracking and the narrower the flooring timber, the less is the movement per board and the easier it is to control. For both strip flooring and laminated floors, the floor boards must not be glued together with a rigid adhesive, because this prevents small shrinkage gaps occurring between each board. Hence, when shrinkage occurs, a few very large gaps can appear, sometimes as very wide splits in a few boards. Some polyurethane floor finishes can cause this problem, so surface finishes should either be applied to each board before laying, or not be applied until after the floorboards have reached their EMC.

Wood flooring on concrete slabs

Dry timber flooring or lining is at greater risk of movement in new construction when moisture from the drying of fresh concrete swells the wood material. This is a particular problem if the timber flooring is glued to a concrete slab or suspended above it. Construction time lines and site protection of dry wood materials become critical in these situations.

Heated floors

Electrical and water reticulated floor heating systems lead to very low moisture contents in timber flooring and, importantly, in floor framing. Designers should specify extra low moisture contents for framing and flooring and seriously consider engineered I-joists for these systems to minimise movement.

Movement of interior timber

Interior panelling

For interior timber panelling, some profiles are better able to maintain appearances with moisture content changes than others. For example, shiplap panels have a flat-bottomed groove between pieces that widen with shrinkage or narrow with swelling whereas tongue, groove and vee panels (T, G & V) have a V-joint which makes movement less conspicuous. Internal panelling should be dry when delivered and kept dry during construction. Moisture content can become as low as 6% in the pitched ceilings of heated houses in winter. Wide profiles will shrink more than narrow ones, and shrinkage movement should be calculated to ensure that tongues are long enough to accommodate the movement. Paint or varnish should be applied before installation, or otherwise it can stick the panelling together, resulting in splitting when shrinkage occurs.

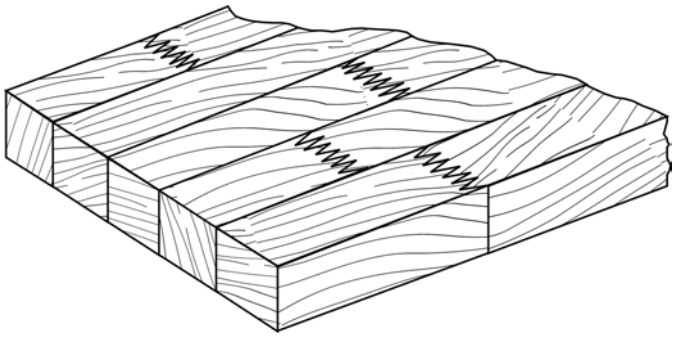

Figure 10 Finger-jointed, laminated timber bench top

Wood-based interior panels such as plywood, particle board or fibre board are normally below EMC at the time of delivery. Typically they will expand during installation. Allowance for this expansion can be made using gaps between panels, gaps along the edges of walls and ceilings or proprietary PVC jointing strips.

Mouldings and joinery

Mouldings and joinery should be dried close to EMC before machining. Normally they will be supplied dry, and must be kept dry on site. Finger-jointed and glue laminated window joinery, doors. sills, etc. give much improved stability over sawn timber because the random effect of each piece 'pulling' in different directions maintains a straight line (**Figure 10**). Mitred joints should be made only in dry timber at the correct EMC, otherwise shrinkage will open up the joints, as shown in **Figure 11**.

Fixings

Detailing must always permit cross-grain movement whether or not the timber is protected by paint or stain. Interior finishing timber up to about 150 mm wide may tolerate double nailing or fixings. Any item wider than this should be nailed or screwed to permit movement and should not be glued. Bench or table tops should be screwed and slotted to allow for this movement.

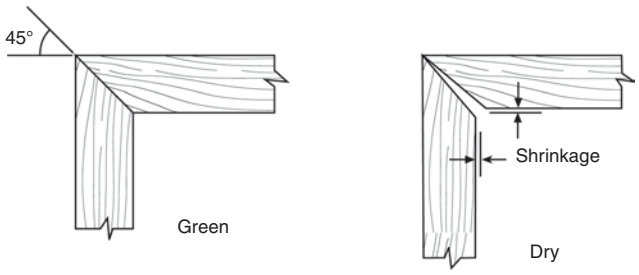

Figure 11 Distortion in mitred joint due to shrinkage

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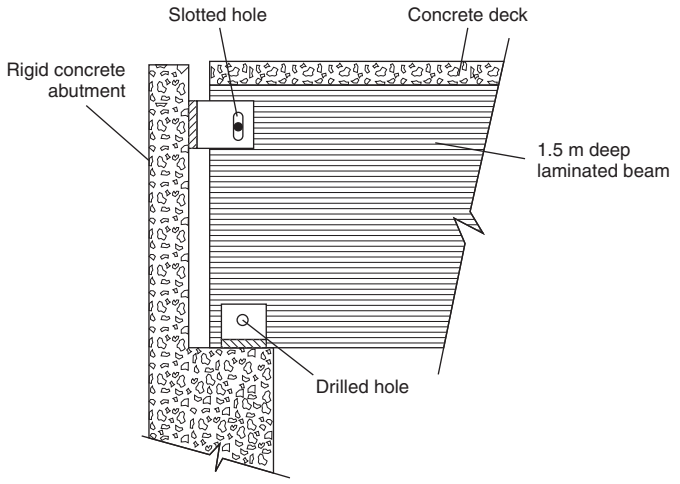

Figure 12 Slotted hole near top of glulam beam to allow for shrinkage movement

External timber

Decking

Exterior decking is frequently used without paint or other applied finish and is most commonly treated pine or imported hardwood. Specifiers are advised to check the species, and calculate the expected movements before detailing sizes of boards and gaps between the boards.

Large bridge beams

Moisture movement in large members exposed to the weather can be significant. For example, a beam laminated at 14% m.c. and placed in service as a bridge beam may attain 20% m.c., a change of 6%. If the beam is 1.5 m deep, transverse swelling is calculated from **Figure 4** as 15 mm. Any connections to abutments or rigid steel components need to accommodate this movement. A possible detail is given in **Figure 12**.

Round timbers

Poles will nearly always check or split longitudinally from the circumference to somewhere near the centre of the pole. Sometimes these checks are distributed around the circumference of the pole, but often there is a single large split. These have little or no effect on the strength of the pole and there is no evidence to suggest decay occurs as a result of such checking in pine poles which have adequate preservative treatment. At the ground line, where decay organisms are most prevalent, the timber is generally of higher moisture content and the splitting not as severe, if it exists at all. When round timbers are used in log house construction, special allowance must be made for the significant settlement due to transverse shrinkage.

Conclusions

There is a saying among experienced timber designers: 'keep it dry, protect it from moisture, don't let it get wet'. The reason for this is that moisture singularly presents the greatest hazard to successful design and satisfactory long-term performance of a timber structure. Understanding the basic nature of timber and how it responds to changes in moisture content is the key to developing good design practices. Designing timber structures to be durable embodies both the art and science of engineering – it is not simply specifying a durable timber or a level of preservative treatment, or even using a reliability based predictive model. Rather it requires a fundamental understanding of timber as a material and an approach that 'designs durability' into the details of a structure, whereby the use of modelling tools and material specifications is part, not all of the final solution.

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Chapter 63

Understanding timber structural connection systems

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The strength and stability of any timber structure depends heavily on the connections between the structural members. One prime advantage of wood as a structural material is the ease with which wooden structural parts can be joined together, using a wide variety of connecting systems. This chapter discusses how the characteristics of timber as an engineering material affect the performance and hence design of connections in timber structures.

Overview of connection types

Conceptually, connections and joints for structural timber can be created in three different ways: (1) with the so-called traditional timber connections used in historical construction, relying on traditional timber joinery notches, lap-joints, and mortise-and-tenon joints transferring the stress directly from member to member, with the additional possibility of wooden dowels, wedges or pegs to provide stability; (2) with metal mechanical fasteners, such as nails, spikes, staples, drift bolts, screws, lag screws and bolts, that usually transfer the stress by shear through the fastener from one member to the other; and (3) with metal connectors, such as plates (split-ring, shear-plate and toothed-plate connectors) and specially shaped steel supports (joist hangers, column bases, framing anchors, etc.), that transfer the load from one member to the other through the metal and the fastener (bolts, nails, etc.) that secure the metal piece to the wood member (Herzog *et al*., 2004). Nowadays, wood members are connected almost exclusively with the latter two systems and a large number of proprietary plates and metal supports exist. For utmost rigidity, strength and durability, each type of connector requires joint designs adapted to the strength of wood along and across the grain, and to dimensional changes that may occur due to fluctuations in moisture content.

More recently, a new type of timber connector has emerged, based on the use of adhesives. Adhesives have been used for some time in the manufacture of engineered wood products, such as plywood and furniture, but they are also becoming very common for structural connections. Structural adhesives are potentially very efficient connectors, since they enable full utilisation of the timber section without the need for holes which remove timber (as in bolted connections), they eliminate the problem of splitting of the grain caused by intrusive fasteners, such as nails, screws, etc., provide a more uniform distribution of stress over the joint area and, aesthetically, they can provide an invisible joint. Currently, their only major limitation is in existing legislation, whereby, they cannot yet replace mechanical fasteners entirely in primary structural connections.

This chapter will describe both mechanical and adhesive based connections. The discussion on mechanical connections focuses on the connector selection, capacity and in-service factors that influence their durability and performance. A more comprehensive discussion of fasteners can be found in *Mechanical Connections in Wood Structures* (ASCE, 1996) and ANSI/AF&PA NDS-2005 *National Design Specification for Wood Construction* (American Forest & Paper Association, 2005). Glued connections are still relatively new and present an innovative and alternative technique to traditional connections. Because of this, basic details of the adhesion phenomenon are presented, along with description of the most common adhesive types, factors affecting their strength and durability, relevant standards for testing and performance assessment, health and safety and quality control issues, and, finally, reference to several sources of further information.

Mechanical connections

Traditional mechanical fasteners have been used since the earliest stools and trestle tables were fashioned from wood. Initially, timber dowels or wedges were used to secure connection details, whether for semi-structural purposes such as for supporting or locking joints in wooden furniture, or for fully structural joints in timber frames. The latter, tended to be replaced with wrought iron pegs or nails, as the knowledge for shaping iron spread (ASCE, 1996).

Today dowel-type fasteners are employed in many situations, typically as part of traditional carpentry methods. For example, in a pinned mortise and tenon, the dowel is used both as a locator and a mechanical locking device, relying on its shear capacity and on the embedding strength

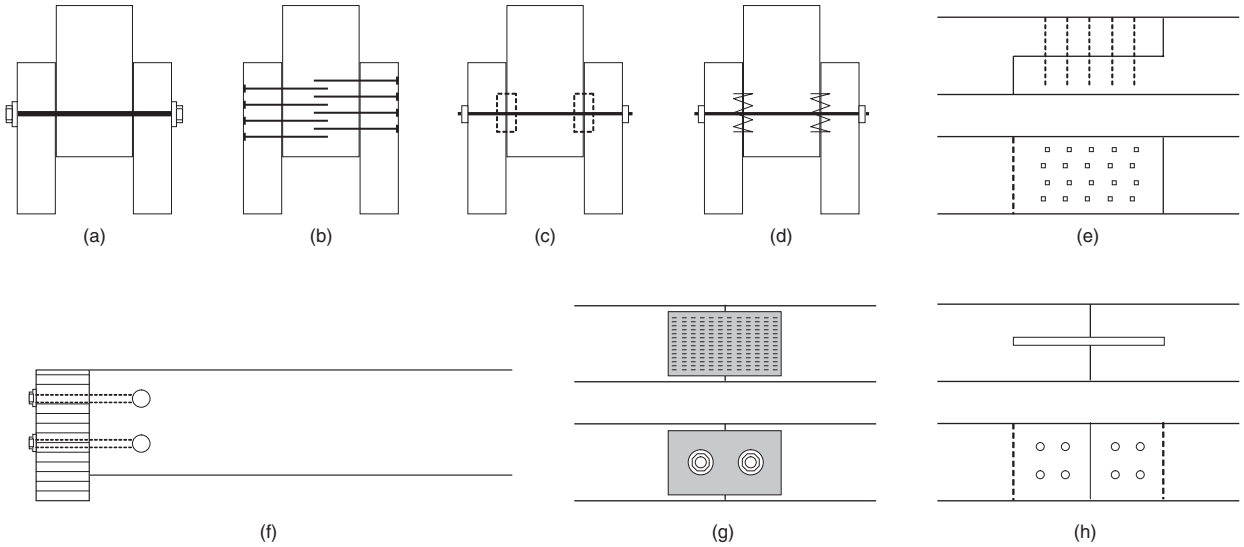

Figure 1 Illustration of various mechanical connection details: splice joint covered by bolted (a) and nailed (b) wooden plates; splice joint with internal shear connectors, split-ring (c) and toothed-plate (d); nailed scarf joint (e); bolted end joint with steel channel (f); splice joint covered by nailed (top) and bolted (bottom) steel plates (g); splice joint with slotted metal plate and dowels (h)

of the wood to resist separation of the timber elements. In lightly loaded beam-to-column joints, dowels are typically made from hardwoods due to their superior shear strength (*Timber Engineering*, 1995).

Many traditional repairs to decayed or damaged timber elements require the splicing of new solid timber sections, often with overlapping timber or metallic plates and clamps. However, it is important to note that in the case of repairing rotten or damaged structural timber, using traditional mechanical or carpentry techniques, the strength of the original timber section is seldom recovered and the appearance and authenticity of the original timber is inevitably lost (*Timber Engineering*, 1995).

Contemporary mechanical connections are constructed using two general fastener types; dowel-type fasteners and bearing-type connectors (split ring and shear plate connectors) as shown in **Figure 1**. Dowel-type fasteners can be designed to transmit either lateral or withdrawal loads (ASCE, 1996). Lateral loads are transmitted by bearing stresses developed between the fastener and the timber. Withdrawal loads are axial loads parallel to the fastener axis transmitted through friction or bearing against the connected timber.

Dowel-type fasteners include dowels, nails, wire spikes, bolts, staples and screws.

- Nails are perhaps the most commonly known mechanical fastenings used in wood construction and as such there are many types, sizes, and forms.
- Wire spikes are generally manufactured in the same way as common wire nails. They tend to have either a chisel point or a diamond point and typically can be of any length up to ~30 cm.
- Drift bolts and pins are long unthreaded steel rods that are driven in pre-bored holes for lateral connections in large timber members. Drift bolts have a head for use with steel side plates and convenience in driving, while drift pins have no head and both are typically inserted using a simple hammer.
- Staples are normally available in clips or magazines for use in pneumatically operated portable staplers and as such allow for rapid construction.

In cases where it would be difficult to fasten a bolt or where a protruding nut on the surface would not be appropriate lag screws are commonly used. These have a hexagonal-shaped head and are tightened by a simple spanner (as opposed to wood screws, which have a slotted head and are tightened by a screwdriver).

Wood screws have flat, oval or round heads. They are usually made of steel, brass, other metals or alloys, and may have specific finishes such as nickel, blued, chromium or cadmium. They are classified according to material, finish, shape of head, and diameter or gauge of the shank. Modifications include self-tapping screws, which have threads the full length of the shank (*Timber Engineering*, 1995).

Bearing-type connectors only transmit lateral loads but provide the highest lateral strength of all fasteners because of the large bearing area provided by the connector. Bearing-type connectors, such as shear plates and split ring connectors, transmit shear forces through bearing on the connected materials (ASCE, 1996). Typical materials employed are mild steel, iron, ceramic, or fibreglass rings or plates placed between members and partially inserted into each adjacent member held together by a bolt, lag screw, or spike grid.

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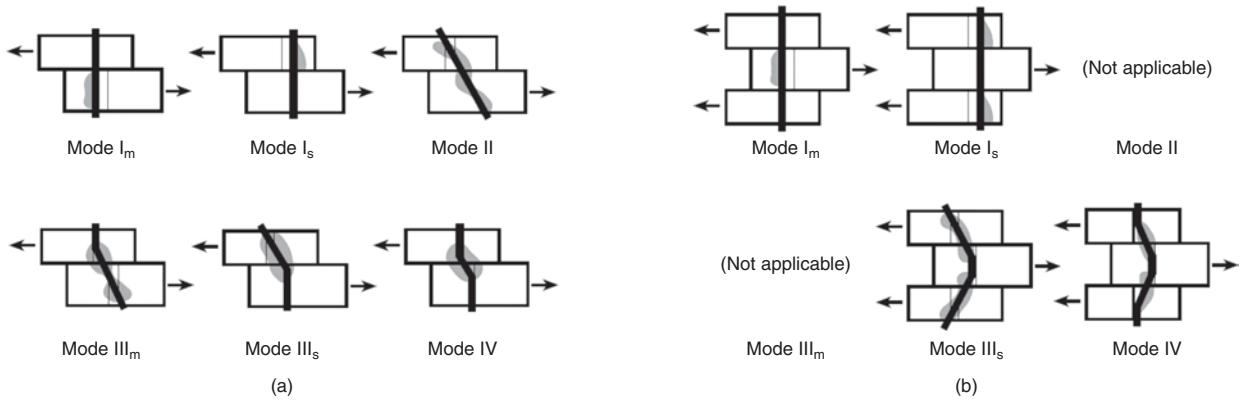

Figure 2 Various combinations of wood-bearing and fastener-bending yields for (a) two-member connections and (b) three-member connections (courtesy of USDA)

Split rings and shear plate connectors are used in conjunction with dowel-type fasteners to resist both lateral loads and bearing loads. They are used extensively in wood trusses and are proprietary products made of light gauge structural quality steel, with zinc or zinc–aluminium alloy coatings, or stainless steel. Various examples of the main connector details can be found in (ASCE, 1996; *Timber Engineering*, 1995; HIS, 1985).

Fastener selection

The selection of a fastener depends on the type of connection and the required strength capacity. Each connection must be designed to transmit forces adequately and provide satisfactory performance for the life of the structure. The basic design procedures for connections are similar to those for structural components. As such, the relevant design codes and standards should be followed, but, in general, for a given connection and type of fastener, the designer must establish:

- design loads
- load-carrying capacity of the fasteners
- withdrawal and lateral resistance of connection
- minimum cross-section of the timber members
- edge and end distances
- minimum spacing in the direction parallel and perpendicular to grain (*Timber Engineering*, 1995).

The capacity of the fasteners themselves is specified by the appropriate design standards and includes yield and ultimate tensile strengths. The strength of the resulting connection is normally limited by either the withdrawal resistance of the fastener, its lateral bearing capacity or bending resistance. In general, these are a function of various empirically derived expressions that are a function of the specific gravity of the timber, the fastener's depth and/or diameter. In addition to the above, **Figure 2** shows a schematic representation of the failure modes applied to the yield model theory (YMT) which selects the worst case of yield modes based on different possibilities of wood bearing and nail bending (nail head effects are not included). The yield strength of these different modes is determined from analytical equations that assume the wood and the bolt are both perfectly plastic. The yield mode that results in the lowest yield load for a given geometry is the theoretical connection yield load. The equations are equally valid for wood or steel side members, which is taken into account by thickness and dowel bearing strength parameters.

Mode I is a wood-bearing failure in either the main (m) or side (s) member; mode II is a rotation of the fastener in the joint without bending; modes III and IV are a combination of wood-bearing failure and one or more plastic hinge yield formations in the fastener. The same equations are also valid for various load-to-grain directions (Department of Agriculture, 1999).

The influence of the grain angle on timber stiffness and strength is an important consideration since, due to the non-isotropic nature of timber, it can contribute to increased stress concentrations if not properly assessed during design, leading to premature failure or material degradation within these highly localised regions. For example, in the connection of secondary beams, the nails/bolts should span over more than 70–80% of the depth of the main beam to prevent possible shrinkage perpendicular to the grain (**Figure 3(a)**). If very stiff plates, located on the upper and lower fibre, are used to join timber elements, care has to be taken to prevent excessive splitting forces developing from tangential forces set up in the fasteners, perpendicular to the grain, due to any slippage or rotation of the connection (**Figure 3(b)**). Dimensional changes due to changes in moisture content can also produce excessive tangential splitting forces if shrinkage is not allowed for in the connection design (*Timber Engineering*, 1995).

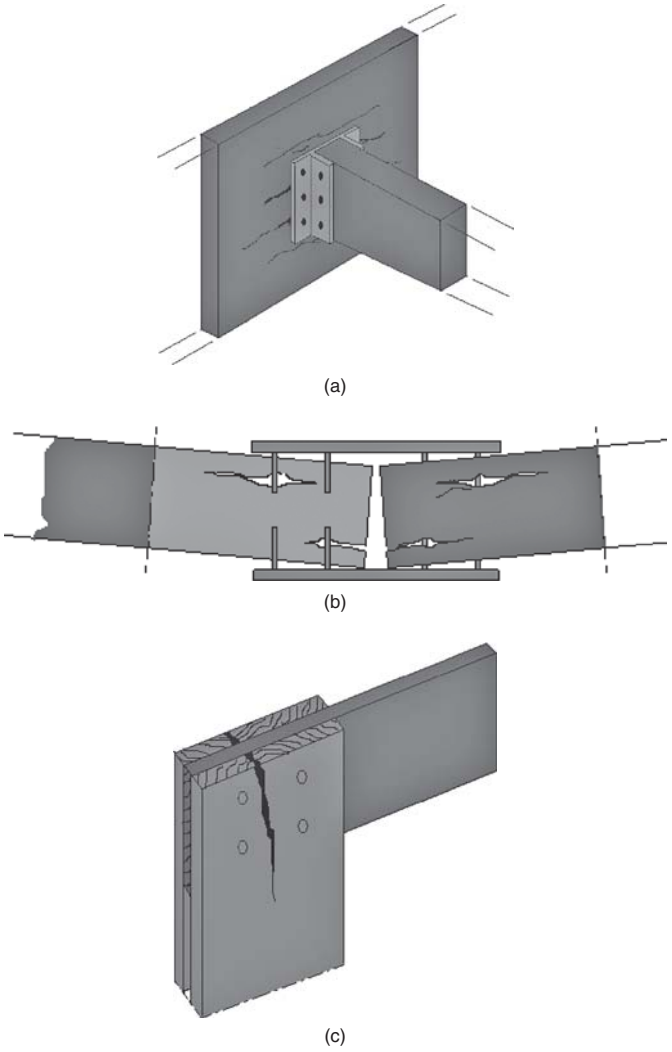

Figure 3 (a) main beam splitting due to secondary beam connection; (b) tangential splitting; and (c) end-grain splitting

Factors affecting in-service performance

In addition to the mechanical aspects of the design, there are specific in-service factors that require careful consideration before the appropriate connector material can be determined, these include:

- temperature exposure
- creep, often combined with the above
- ultra-violet (UV) exposure
- moisture content
- chemically aggressive environments
- fatigue resistance
- corrosion (galvanic and contact).

The above factors' influence on the mechanical performance of the connection varies considerably depending upon the materials used in the manufacture of the mechanical fasteners, on timber species and preservative treatment. The common metallic materials include iron, wrought iron, steel, stainless steel, zinc-coated steel, bronze and aluminium. The long-term mechanical behaviour of these materials, such as strength, stiffness and even creep, is well documented. However, corrosion due to in-service factors in combination with timber is less well understood. For non-metallic connectors, e.g. fibre-reinforced polymer (FRP) dowels, plates, rods and bolts, some thought needs to be given to the particular matrix resin of the FRP employed. The most common resin matrices are polyesters, vinyl-esters, epoxies and phenolics, all of which are susceptible to reductions in performance due to moisture ingress (plasticisation), sustained high temperatures (creep), UV exposure (mechanical property degradation) and load-type (creep and/or fatigue) (Murphy, 1994; Hutchinson, 1997; Department of Agriculture, 1999; Cripps, 2002). The bond between the fibre and the resin matrix or the resin matrix itself can also be degraded, either of which results in the degradation of the mechanical properties with load history (Hutchinson, 1997). Additives can be used to improve resistance to the above and there are many examples of glass-fibre-reinforced polymer (GFRP) cladding remaining stable over 25 years (Cripps, 2002), albeit typically employed in an un-stressed state.

The effects of various chemical interactions between adjacent contacting materials can result in gross corrosion or chemical degradation of one or the other. This can take the form of an electro-chemical process (galvanic corrosion) between different conducting materials (moisture being the electrolyte) or chemical attack from extractives in the timber, such as tannins in oak. Clearly, the moisture present in timber can corrode unprotected metallic nails with the formation of the breakdown product iron-oxide (rust). Fasteners embedded within the timber can also suffer from electro-chemical corrosion. Usually, this type of corrosion occurs between two dissimilar metals (bi-metallic corrosion), one exhibiting a more noble form than the other, producing a cathodic and anodic cell. However, single fasteners can also suffer dramatic losses of material in a similar manner if one part of the fastener becomes effectively isolated from the other. This can happen if oxygen is lacking around a relatively small part of the fastener or the fastener is in contact with timber containing two different acid or salt contents, e.g. the fastener joins two different timber species (IHS, 1985). The key factors involved in the corrosion of metallic fasteners are therefore:

- moisture
- the presence of salts
- wood preservatives

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Fastener material	Corrosion resistance
Monel	Good
Stainless steel	Good
Bronze	Medium
Aluminium	Medium
Steel alloy	Poor
Iron	Poor
Coating material	
Thermoplastic	Good
Zinc plus paint	Good
Zinc	Medium
Bitumen	Medium

Table 1 Common fastener materials' corrosion resistances (Department of Trade and Industry, 2003)

Timber species	Timber acidity: pH	Timber/vapour corrosion potential
Oak Chestnut	3.3–3.9	High
Beech Birch Douglas Fir Cedar	3.5–5.4	Fairly high
Spruce Teak Pine Elm Mahogany Walnut Iroko	4.0–8.8	Moderate

Table 2 Guide to potential timber acidity according to species (Department of Trade and Industry, 2003)

- flame retardants
- storage near to the sea
- the fastener material, surface coating and condition
- timber species (HIS, 1985).

The presence of moisture is the most significant factor. Maintaining the timber moisture content below that of 18–20% is generally accepted as sufficient to prevent any significant corrosion of the fixing (HIS, 1985).

In the case where moisture is acting as the electrolyte solution, its conductivity itself is highly dependent upon its salt content; more and more the corrosion rate increases. Salt is found in various wood preservatives, flame retardants and timber having been transported or stored in or near seawater. Wood preservatives fall into three main categories: creosote/tar-oil, organic solvents and water-based solvents (HIS, 1985). They are usually employed to prevent insect or fungal attack brought about by high levels of moisture. According to HIS (1985) the creosote-based products and organic solvents are relatively benign, and if anything, help to reduce corrosion as they form a barrier to the migration of moisture. Some waterborne solvents such as copper-chrome-arsenic (CCA) can increase the corrosion of metallic surfaces such as zinc and aluminium. Soluble sodium sulfate, formed during the curing reaction of the salt formulation, enhances the conductivity of the surrounding electrolyte solution. In this process, copper salts are left in the timber, which can then form a bi-metallic cell with the zinc or aluminium fastener materials. The use of CCA and creosote has been banned and organic solvents have been restricted in Europe (European Union Commission Directive 76/769/EEC). Flame retardants also contain inorganic salts, such as ammonium phosphate and ammonium sulfate (HIS, 1985). These can be highly corrosive in comparison to other inorganic salts and boron salts that are also used for flame retardants, which tend to be far less aggressive.

The type of metallic fastener (including its surface coating and condition) also plays a key role in the rate of development of corrosion products. The common materials used for mechanical fasteners include iron, wrought iron, steel alloys, stainless steel, coated steel (traditionally using bitumen, grease and lanolin but more recently using chlorinated rubber, epoxy and zinc), silicon bronze, phosphor bronze, aluminium, and monel (30% copper–67% nickel). **Table 1** provides a simple guide to the corrosive resistance of certain fastener materials and their coatings.

A separate source of corrosion is found due to the contact of the fastener with acidic solutions present in the timber. The natural chloride content of timber is low (e.g. 0.2 to 4% of dried ash) and various timber species exhibit quite high levels of acidity, as indicated by the pH values given in **Table 2**. The majority of acidity is derived from the production of acetic acid, which, as a percentage of the dry wood, can range from 1 to 6%, depending upon the timber species. Acetic acid is one of the products (along with hydroxyl radicals) formed when cellulose hydrolyses in the presence of moisture (Department of Trade and Industry, 2003). It is also important to note that the formation of acetic acid is dependent upon the moisture content and temperature of the wood, and the overall geometry of the timber section. This means that, depending upon the species, acetic acid production can continue over many decades and, as such, the pH values of timber should be taken as only indicators of the potential for corrosion.

A further concern for timber structures and the appropriate selection of fastener material is their location.

Design considerations	Conditions	Timber service class	Potential risk of corrosion		
			Low	Medium	High
Life of structure	Semi-permanent	I or II		X	
	Permanent				X
Exposure	Indoor, heated	I	X		
	Covered, unheated	II		X	
	Open				X
Corrosivity of wood (**Table 2**)	High or fairly high	I or II			X
	Moderate			X	
Situation (external)	Within 1 km of coast	II			X
	1–5 km of coast			X	
	Inland		X		
Wood treatments	Salt seasoning	II			XX
	Fire retardant	I or II			X
	Inorganic preservative, young	II		X	
	Inorganic preservative, aged		X		
	Organic preservative		X		

Table 3 Guide to potential risk of corrosion related to timber service class I or II (Department of Trade and Industry, 2003)

Table 3 provides a general guide to the various design considerations for the assessment of potential risk of corrosion to structures depending upon location and wood characteristics (Department of Trade and Industry, 2003).

Material testing for measuring corrosion of metals in wood

According to Zelinka and Rammer (2005) there is only one current standard to assess the corrosion of metals in wood, developed by the American Wood Preservers' Association (Wood Protection Association, 1994). Although this test provides a rapid assessment, it is unclear how the measured corrosion rates relate to performances at ambient temperatures and humidities of wood in service (Zelinka and Rammer, 2005, 2006). More information on determination of corrosivity of fasteners can be found in these references.

Summary

Various fasteners and connectors are available to the designer. Design of wood joints should be carried out according to Eurocode 5 taking into account the design loads, fastener characteristics, and wood properties, as well as environmental factors/exposure conditions. However, specific details to ensure durability of a connection are generally not codified and must be decided by the designer.

Corrosion-resistant fastener materials, such as stainless steel, monel, aluminium and copper-based alloys should be employed in cases where it is known there is a high moisture (>20% MC), salt or acidic presence. Where costs prevent the use of the above, galvanised steel can be used in situations where experience has indicated this to be satisfactory, otherwise alternative protective coatings should be explored. UV protection, fatigue and creep need special consideration where FRP materials are potentially exposed to sunlight and/or high temperatures.

Wherever possible, connector plates should be insulated from the timber. This could be achieved via various coating materials but also, for example, through the employment of plastic shims, plastic sleeves, bonded GFRP barrier layers or bitumen felt. Often external structures are painted, providing a further barrier to moisture. In this case it is important to maintain the paint layer and repair any blistering or peeling, which can cause entrapment of moisture.

The joining of different wood species using metallic fasteners should be undertaken with caution and different fastener materials that are in direct contact with each other should be avoided wherever possible.

Finally, where corrosion is considered to be a critical factor in the design or repair, which is a function of environmental exposure (timber class), timber species and presence of preservatives, tests should be conducted on the specific materials under consideration. However, to date there are very few appropriate test methods available, although explorative tests using Electrochemical Impedance Spectroscopy exhibit good potential.

Glued connections

Introduction

The widespread use of adhesive bonding can be attributed to its inherent advantages, which include its ability to distribute the applied load over the entire bonded joint area resulting in a more uniform distribution of stress (compared with mechanical connections), it requires little or no damage to the adherends, adds very little weight to the structure, generally has a superior fatigue resistance to

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Figure 4 Joints of equivalent load capacity with various connectors

other joining methods, is suitable for joining dissimilar materials, and can reduce manufacturing costs.

The structural advantage of adhesive connections is conveniently illustrated by the joints prepared by R. A. G. Knight back in 1959, formerly Head of the Composite Wood Section at the Forest Products Research Laboratory, UK (Adams *et al.*, 2004). Each joint shown in **Figure 4** was designed to take the same working load of 15 kN, based upon empirical rules and experience. The adhesive was resorcinol-formaldehyde, which was stated to be fully weather resistant and would outlast the wooden adherends. This is in contrast to the mechanical connections, which would either corrode or degrade the wood without additional protective coatings or treatment. The mechanical connectors included bolt and a spreader plate, bolts, and nails. Clearly the bonded joint is significantly more efficient in terms of the joint area and weight.

An important application where bonded structural timber joints are particularly well suited is in the repair and upgrade of existing, and often historical, timber structures. These types of applications provide a means of repairing timber structures in situ with minimal intrusion into the existing original structure. The repair is often invisible and can even be shaped to disguise the appearance of the modern repair to suit that of the original structure. In some cases there is enhanced fire protection, due to the connection detail being completely encased within the timber, and, perhaps most importantly, repairs for historic buildings can be removed or replaced in the future whilst still retaining an enhanced amount of original timber compared to a repair done by a traditional carpentry technique. Typical methods include using steel or FRP rods, either bonded into slots or drilled holes, to connect two timber sections (**Figure 5**) or bonded-rods into transversely drilled holes through a single timber section to prevent delamination (glulam) or splitting.

The major disadvantage in the use of adhesive connections is primarily their assembly, requiring appropriate care of the joining surfaces prior to bonding, suitable surface preparation or pre-treatment, time to cure and a

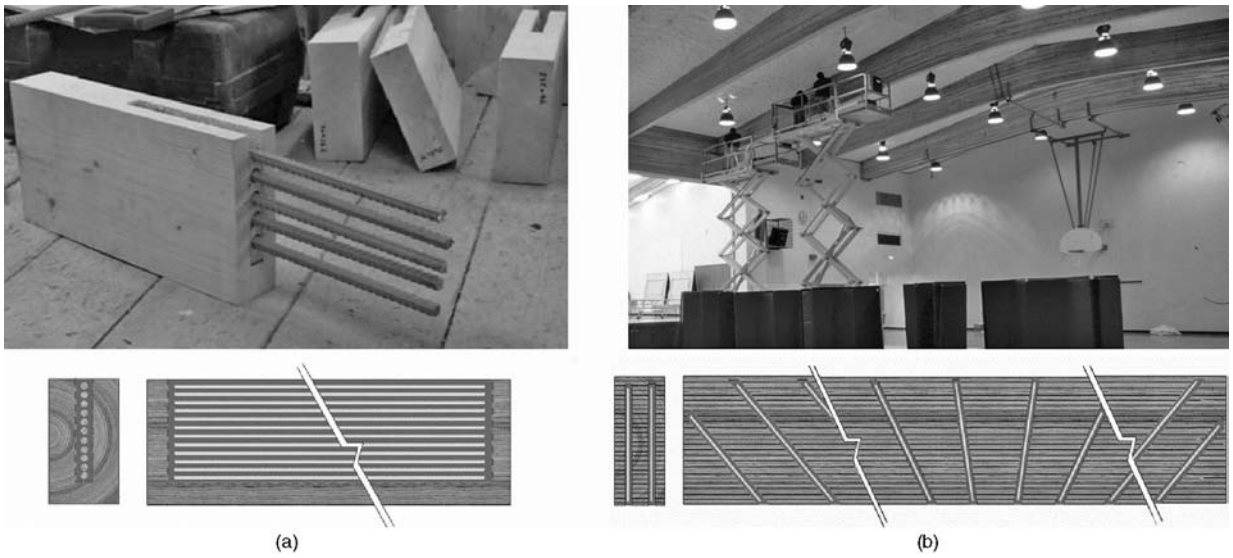

Figure 5 (a) Bonded-in steel rod connector for rotten beam-end repairs and schematic of typical repair; (b) bonded-in steel rod connector for upgrading glulam members and schematic of typical repair (courtesy of Rotafix Ltd)

means of holding the joint together until cure is complete. It is important to recognise that all of the above requires a fully trained and relatively skilled operative, which contrasts with mechanical connections that are often much simpler to prepare and, of course, much more commonly used. Adhesive joints must also be designed to eliminate peel and cleavage stresses; some adhesives also have limited elevated temperature resistance, they may be affected by service environments and are 'perceived' to be difficult to disassemble for inspection and repair (Department of Agriculture, 1999; Aicher *et al.*, 1998; Plecnick and Bresler, 1980).

The greatest hindrance to the exploitation of structural adhesive bonding of timber is the lack of suitable design guidance (and guidelines) together with a general lack of experienced personnel, whether engineers, designers, contractors or clients. Current European design guidance is limited to that provided in Eurocode 5 (CEN, 2004) and the *Timber Engineering STEP 2* manual (1995). Recently, there has been much activity under the COST Action E34 to explore bonding of timber on site, the results of which were recently published and can provide additional guidance. Other assistance can be readily found at http://www.licons.org, and although this work is specifically aimed at conservation, there is plenty of information relating to the general application and use of structural adhesives for bonding timber.

Adhesion to wood

Gluing wood successfully depends on how well we understand and control the complexity of factors that constitute the individual links in the adhesive bonding of wood components. The development of new kinds of structural adhesives and composites plus the great diversity in the shapes and species of wood elements being glued have combined to produce a continuous source of challenges and problems, leading to an increase in the knowledge about adhesion and cohesion of adhesive compounds (Pocius, 2002).

Marra (1992) divides gluing technology into two parts: bond formation and bond performance. In his analysis, bond formation is related to the fluid properties of adhesives (required to allow the proximity necessary for adhesion to take place) and their conversion to solids. Bond performance involves the solid properties of adhesives and how they react to stress and degrading agents, such as heat and moisture. In view of the fact that the performance of the bond is also affected by how the bond is formed, the two parts are co-dependent. Besides that, due to the stresses that are created and transferred in and across the glue lines, generally in a non-uniform way, it is possible that the bond will perform satisfactorily but the final glued product will not, and that's why it is necessary to consider a third part, the product performance. To achieve this, Marra considered the gluing process as a sequence of events, and identified factors that can be grouped in different materials and operations. Each group can then be considered as representing a distinct area of activity, all contributing accumulatively to the final glued product performance. The first four groups in Equation (1) affect bond formation, the fifth group affects both bond formation and bond performance and the last two affect bond performance only.

$$\begin{aligned}\text{Glued product performance} = {} & \text{Potential adhesion forces} \pm \text{Adhesive composition factors} \pm \text{Wood property forces} \\ & \pm \text{Wood preparation factors} \pm \text{Adhesive application factors} \\ & \pm \text{Wood geometry factors} \pm \text{Product service factors}\end{aligned} \quad (1)$$

From the instant an adhesive is applied to wood until the adhesive cures, several important steps must take place for a good bond to occur. In the majority of wood gluing processes, the adhesive is applied to the surface of one of the adherends, and then a second adherend is pressed into the adhesive. It is important that the adhesive properly flows to uniformly cover both wood surfaces. Ideally, the adhesive should penetrate the wood surface, filling the small voids caused by pores, checks and other anatomical features. If the adhesive is fluid enough it can penetrate into and between the wood fibres where it will be locked after curing (mechanical entanglement), however in practice the adhesives penetrate only two or three layers of fibres, i.e. a few hundredths of a millimetre (Department of Agriculture, 1999).

While penetration allows the adhesive to contact the wood deep into the crevices on the wood surface, wetting involves the attraction of the liquid adhesive to the wood surface via molecular forces. These intermolecular forces appear only when the adhesive molecules come very close to the wood cellulosic molecules (Pizzi and Mittal, 2003).

Van der Waals bonds (dipole to dipole, dipole to induced dipole and dispersion forces) provide bond energies up to 40 kJ/mol, which are low compared to true chemical bonds, but it may be enough to achieve a good bond. Wood has a polar surface and thus polar adhesives will be attracted to its surface, for example urea-formaldehyde (UF) resins, isocyanates from the polyurethanes (PU) adhesives, epoxies, vinyl adhesives, etc.

Hydrogen bonds may also form because the cellulose molecules in the wood contain hydroxyl groups (−OH) and if the adhesive also contains hydroxyl groups (such as

phenolics, epoxies, UF, RF, PU, etc.) these will build hydrogen links between the proton-like character hydrogen and the nucleophile oxygen atoms. Hydrogen bonds do not exceed bond energies of ~30 kJ/mol.

True chemical bonds are the strongest bonds and can be obtained when a chemical reaction occurs between the cellulose and the adhesive molecules, for instance with PU, epoxies or isocyanates. Covalent bonds can provide bond strengths in the order of 80–1100 kJ/mol.

Adhesion is a very complex phenomenon and is therefore beyond the reach of a single model or theory. The study of adhesion involves several scientific fields, including macromolecular science, physical chemistry of surfaces and interfaces, materials science, rheology, mechanics and micromechanics of fracture. At present, no practical unifying theory describes all adhesive bonds. The most consensual theoretical models of adhesion so far proposed are: (a) mechanical interlocking; (b) electronic or electrostatic theory; (c) theory of weak boundary layers and interphases; (d) adsorption (thermodynamic) or wetting theory; (e) diffusion theory; and (f) chemical bonding theory. Currently (2009), each of these theories is valid to some extent, depending on the nature of the adherends and the conditions of formation of the bonded system. In practice, several of these mechanisms may be occurring simultaneously. Of the various mechanisms existing, it is now commonly accepted that the one with the widest applicability is the adsorption or thermodynamic theory. This theory is based on the belief that adhesion will occur between the adhesive and adherend because of physical forces established at the interface, as long as an intimate contact is achieved. The intimate contact step is necessary for interlocking, interdiffusion and chemical bonding mechanisms to occur. The magnitude of these forces can be related to fundamental thermodynamic quantities, such as surface free energies of both adhesive and adherends (Pocius, 2002; Pizzi and Mittal, 2003).

The extent to which interatomic and intermolecular forces develop between adhesive polymers and wood adherends is uncertain. A number of people have speculated that covalent bonds occur between the adhesive and the wood, but the proof is weak, and considerable controversy still persists about their presence (Pizzi and Mittal, 2003). Also, because of wood's porous nature and its pronounced surface roughness, mechanical interlocking and valence forces (van der Waal's forces, London forces and hydrogen bonding) are now accepted as the main mechanisms by which bonds between adhesive polymers and molecular structures of wood are formed (Department of Agriculture, 1999; Working Group 1, 2002).

The strength of a bonded joint can be expressed as a function of three terms: the interfacial molecular interactions; the mechanical and rheological properties of the bulk materials and the characteristics of the interface. In the last decades a great deal of attention has been given to the first two terms, as a result of studies in the physical chemistry of surfaces and fracture mechanics. But the last term still lacks proper and complete understanding (Pocius, 2002; Pizzi and Mittal, 2003).

Adhesives types

There are many ways of classifying adhesives. In this section they will primarily be grouped as structural, semi-structural or non-structural. A structural adhesive has several definitions. For instance, according to the European standard EN 923, it is defined as 'being able of forming bonds capable of sustaining in a structure a specified strength for a defined long period of time'. The Adhesive and Sealant Council has defined it as 'an adhesive of proven reliability in engineering structural applications in which the bond can be stressed to a high proportion of its maximum failing load for long periods without failure'. The ASTM D907 defines it as 'a bonding agent used for transferring required loads between adherends exposed to service environments typical for the structure involved'. Despite the definitions adopted, this group of adhesives typically encompasses those materials which give tensile shear strengths of at least 4 to 10 MPa, and produce joints of such strength and durability that the integrity of the bond is maintained in the assigned service class throughout the expected life of the structure. Semi-structural adhesives generally do not withstand long-term static loading without deformation. A non-structural adhesive is used in applications that generally require only low strength or temporary fastening.

Independently of the structural type, the adhesive should be able to maintain integrity of the assembly under the expected conditions of service. Therefore, several service classes have been established in order to separate adhesives according to their durability. Standards such as BS EN 1995-1-1: 2004, UK National Amendment to BS EN 1995-1-1: 2004, BS 5268-2: 2002, BS EN 301:2: 2006 give the types of environments that an adhesive can be subjected to service (**Table 4**).

Table 5 presents the most common wood adhesives categorised according to their expected structural performance at varying levels of environmental exposure.

As discussed previously, a broad array of adhesive types is available for bonding wood materials. These adhesives were classified earlier by their relative strength and durability, but they can also be classified according to the chemical and physical nature of the adhesive resin, as either synthetic or natural (Pizzi, 1983, 1989; Mays and Hutchinson, 1992; Department of Agriculture, 1999; Pizzi and Mittal, 2003; EN 923: 2005). We can further categorise adhesives by whether or not they form cross-linked

Temperature	Climatic equivalent	Examples	EN 1995	BS 5268	EN 301
>50°C	Not specified	Prolonged exposure to high temperature	1, 2, 3	1, 2, 3	I
<50°C	>85% RH at 20°C	Full exposure to weather	1, 2, 3	1, 2, 3	I
	<85% RH at 20°C	Heated and ventilated building	1, 2	1, 2	II
		Exterior protected from weather			
		Short periods of exposure to weather			

Notes: 85% relative humidity (RH) at 20°C will result in a moisture content (MC) of ca. 20% in softwoods and most hardwoods, and a somewhat lower MC in wood-based panels. In service class 1 the average MC in most softwoods will not exceed 12%. In service class 2 the average MC in most softwoods will not exceed 20%. In service class 3 the average MC in most softwoods can exceed 20%

Table 4 Adhesive types for use in different climatic conditions

molecules (i.e. thermosetting or thermoplastic), by their flexibility (i.e. rigid or elastomeric), and by their form and curing conditions.

In the wood and building industry a great variety of adhesives are currently in use, and their development is driven by adhesive cost-reduction, faster processing time, and specialised products where complex adhesive formulations are required. The basic chemicals most commonly used for wood adhesives and resins are formaldehyde, urea, melamine, phenol, resorcinol and isocyanate. However, despite the apparent simplicity, in terms of families of chemicals, the formulations are highly complex mixtures of chemicals and additives. Various wood adhesives, along with their typical applications, are presented in select references (Sellers and McSween, 1988; Department of Agriculture, 1999; Conner, 2001; Buschow, 2002).

Adhesive selection

The adhesive selection process is difficult because many factors must be considered, and there is no universal adhesive that will be adequate for every application. The various adhesive families overlap somewhat with regard to their characteristics and can provide similar properties, though there is also a significant variation within certain adhesive families. It is usually necessary to compromise when selecting a practical adhesive system. Some properties and

Structural integrity	Service environment	Adhesive type
Structural	Fully exterior (withstands long-term water soaking and drying; equivalent to EN 1995-1-1 service classes 1, 2, 3)	Phenol-formaldehyde, PF
		Resorcinol-formaldehyde, RF
		Phenol-resorcinol-formaldehyde, PRF
		Emulsion polymeric diphenylmethane diisocyanate, eMDI
		Melamine-formaldehyde, MF
	Limited exterior (withstands short-term water soaking; equivalent to EN 1995-1-1 service classes 1, 2)	Melamine-urea-formaldehyde, MUF
		Polymeric diphenylmethane diisocyanate, pMDI
		Epoxy, EP
		Toughened acrylic
	Interior (withstands short-term high humidity)	Urea-formaldehyde, UF
		Blood
		Casein, CS
Semi-structural	Limited exterior (withstands short-term water soaking; equivalent to EN 1995-1-1 service classes 1, 2)	Polyurethane, PU
		Cross-linked polyvinyl acetate, PVAc
		Polyesters (unsaturated)
Non-structural	Interior (withstands short-term high humidity)	Animal
		Elastomeric construction
		Elastomeric contact
		Ethylene vinyl acetate, EVA
		Polyesters (saturated)
		Polyvinyl alcohol, PVA
		Polyvinyl acetate, PVAc
		Soybean
		Starch

Note: This table presents the common assignment of an adhesive type to a structural/service environment category available in the literature, but it does not eliminate the possibility of using specific adhesive formulations in other categories

Table 5 Typical choices of adhesives in different service environments (adapted from Department of Agriculture, 1999; Pizzi and Mittal, 2003; Mays and Hutchinson, 1992; Pizzi, 1983, 1989)

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characteristics that are desired will be more important than others, and a thoughtful prioritisation of these criteria will be necessary when selecting an appropriate adhesive. One needs to optimize the entire bonding process and not merely a single part of the process. Considerations need to be given at the same time to the substrates, joint design, service stress, service environment, surface pre-treatment, quality control, application and curing methods. However, with the many types of adhesives available and the many available specific formulations that can be provided, a good solution can usually be found.

There are several different methodologies used to select an adhesive for a specific application. One of them consists firstly in considering the structural integrity that is required of the adhesive. After choosing the type of adhesive needed (structural, semi-structural or non-structural) it is time to select the service environment to which the joint will be exposed. These two steps can be achieved consulting a table containing the adhesive's durability categorised into levels of structural integrity (such as **Table 5**). By using these factors as criteria for selection, the many commercially available adhesives can be narrowed down to a few possible candidates. To further narrow down the number of candidates, their strength, durability, preparation, and application characteristics should be analysed (Sellers and McSween, 1983; Department of Agriculture, 1999; Conner, 2001; Dunky and Pizzi, 2002). Once the general product requirements, joint design and service requirements are known, adhesive systems with good durability can be selected. The final step of choosing which specific formulation is best suited for the intended application (as well of establishing the need of surface preparation and additional bonding requirements) should be made in cooperation with the adhesive supplier. The desired form of the adhesive and appropriate method of application can then be chosen based on the formulation possibilities available within the class of adhesive chosen.

Another factor for consideration is the possibility of over-specifying the adhesive requirements. This should be avoided, because requirements for higher strength or greater heat resistance than is actually necessary could exclude many formulations that may be equally acceptable for the job, but at lower cost. Ideally, the objective in specifying an adhesive is to select a material that will cause the joint to fail in the substrate instead of failing in the adhesive.

To select adhesive candidates, it is necessary to have meaningful physical data for comparison. General physical properties such as shear or peel strength can easily be compared. Literature from adhesive manufacturers and trade periodicals offers useful information about the adhesive specifications, uses and strength values. Various sources for such information are provided in the Further reading section.

Strength and durability

Although obtaining high initial bond strength is relatively easy, obtaining good bond durability is comparatively more difficult. The ability of a joint to maintain satisfactory long-term performance, often in severe environments, is therefore an important requirement of a structural adhesive joint, because it should be able to support design loads, under service conditions, for the planned lifetime of the structure (Mays and Hutchinson, 1992).

A number of factors determining the durability of structural adhesive joints have been identified and can be grouped into three categories: environment, materials and stresses. The environment is dominated by temperature and moisture. The materials category includes the adherend, the adhesive and the inter-phase between them both. The last category refers to the stress the bond is subjected during or after exposure to service environment, affecting both longevity and residual strength.

Environmental effects

The environment to which joints are exposed plays an important role in their durability. Moisture and heat are the most important factors in determining the strength loss of a joint exposed to the service environment (Kinloch, 1983; Custódio *et al*., 2008).

Moisture

Water is often regarded as one of the most worrying agents that may affect the properties of an adhesive and the interface between it and the adherend. Most bonded structures, when exposed to water or humidity, will lose strength over a period of time and in rare cases they may collapse, although this effect is limited to extreme conditions (Department of Agriculture, 1999). It is almost impossible to keep water out of an adhesive joint. There are a number of possible mechanisms by which water may enter a bond: (a) diffusion through the adhesive from the exposed edges; (b) transport along the adhesive–adherend interface; (c) migration by way of capillary action along cracks and crazes in the adhesive; and (d) diffusion through permeable adherends, such as composites and wood (Pizzi and Mittal, 2003).

The presence of water may also produce an unstable adhesive–adherend interface, which will gradually be displaced from the adherend surface by water. Ultimately, water may penetrate the adherend surface and produce a loss in the strength of the adherend itself. Stress can enhance the effect of these irreversible processes, because internal stresses produced by swelling will add to the existing externally applied stresses in the joint. The damage produced by these mechanisms will accumulate in the bond, until a threshold state is reached that produces

a rapid propagation of the damage, leading to complete failure of the structural joint (Hartshom, 1986; Gledhil and Shaw, 1990; Pizzi and Mittal, 2003).

In general, the amount of moisture in wood, combined with water in the adhesive, will greatly affect wetting, flow, penetration and cure of aqueous wood adhesives. However, research studies carried out in recent years have proved that this is not true for all adhesives. For instance, Wheeler and Hutchinson (1998) and Broughton and Hutchinson (2001a) found that the epoxy adhesives used were able to bond timber with moisture contents up to 22%, without any significant deterioration in bond strength or change on mode of failure.

Surveys, recordings and long-term monitoring of bonded timber structures undertaken in the last decades, have provided some information on the ways in which bonded timber joints behave. Nevertheless, even with the greater understanding of this technology, doubts over structural performance, the effect of moisture on long-term durability, still persist for some adhesives (such as epoxies, polyurethanes, etc.). Bond enhancement is not required to increase initial strength, but where connections, made with less durable adhesives, may be subjected to repeated wetting and drying, it may be necessary to improve long-term durability of the structural joint with primers, coupling agents and other surface treatments (Back, 1991; Wheeler and Hutchinson, 1998; Broughton and Hutchinson, 2001b, 2008; Leafblad, 2005; Custódio *et al*., 2008).

Temperature

As well as moisture, temperature is also an important factor in the durability of structural adhesive joints, since it can affect the creep, fatigue and fire performance of adhesive joints. It will therefore be necessary to select an adhesive that is appropriate for the in-service environment of the joint.

The effects of moisture and heat, especially in combination with an applied stress, have a considerable influence on the durability of structural adhesive joints. In the short term, the mechanical properties of timber, adhesives and bonded products vary according to the specific environment where they are applied. In general, all properties decrease as the temperature and moisture levels increase. In the long term, wood, adhesives and bonded products will degrade at a rate determined by the temperature, moisture and level of stress (Avent and Issa, 1984; Department of Agriculture, 1999; Leafblad, 2005).

Well-designed and well-made joints with any of the normally used structural woodworking adhesives should retain their mechanical properties indefinitely if the wood moisture content stays below approximately 15% and if the temperature remains within the range of human comfort. However, when adhesives are exposed, either intermittently or continuously, to high temperatures for long periods they will eventually deteriorate (Plecnick and Bresler, 1980; Mays and Hutchinson, 1992; Working Group 1, 2002).

At abnormally high or low temperatures the adhesive in a bonded joint may experience severe internal stresses that develop when different materials within the joint have different coefficients of thermal expansion. Polymers also tend to get soft at elevated temperatures and brittle at low temperatures, and long-term exposure to elevated temperature could also cause their oxidation or pyrolysis (Plenick and Plenick, 1986; Kelsey and Biswas, 1993; Pizzi and Mittal, 2003).

Recent studies have challenged the widely held belief that glue bond is protected by the low thermal conductivity of wood (Aicher *et al*., 1998; Kemmsies, 1999; McMurray and Amagi, 1999; Aicher *et al*., 2002; Cruz *et al*., 2004, 2005; Frangi and Fontana, 2004; Cruz and Custódio, 2006). Therefore, when considering joint strengthening with low T_g (glass transition temperature) adhesives (such as epoxy, polyurethanes, etc.) additional design considerations should be made to account for the predicted service temperature: e.g. shading and ventilation should be adopted to prevent the adhesive from overheating; edge and end distances of the glue lines should be increased whenever possible; and adhesives with suitable T_g higher, at least 10–20°C, than the glue line expected service temperature should be selected (Cruz and Custódio, 2006; Custódio *et al*., 2006, 2007, 2008a, 2008b).

Although, Eurocode 5 deals with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures, it only alludes briefly to adhesives, referring only to phenolic and amino plastic types. In Eurocode 5, only three requirements regarding structural adhesives are made: (1) the adhesives shall produce joints of such strength and durability that the integrity of the bond is maintained in the assigned fire resistance period; (2) PF and amino plastic Type 1 adhesives according to BS EN 301: 2006 may be used in all service classes to bond wood to wood, wood to wood-based materials or wood-based materials to wood-based materials, and also that adhesives according to BS EN 314-1: 2004 may be used for plywood and LVL; (3) adhesives which comply with Type II specification as defined in EN 301 should only be used in service classes 1 or 2 and not under prolonged exposure to temperatures in excess of 50°C.

In summary, the use of structural adhesives should always take into consideration the service temperature effect on adhesive performance. Extensive pre-normative research and thorough consideration of these effects are still required for the development of European standards for the evaluation of bond durability and long-term performance under high service temperature for structural and semi-structural adhesives other than phenolic and amino

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plastic types. This will enable the effective and safe application of techniques based on the use of structural adhesives, especially in highly demanding situations where the present lack of knowledge and reliability of these products hinder their widespread use.

Materials

Besides the environmental factors mentioned above, the materials involved in a structural joint also influence bond strength and durability. The factors in the material category include: the adherends; the adhesive; the design of the joint; freedom from surface contamination (including extractive contamination); stability of the adherend surface; the ability of the adhesive to wet the surface; and entrapment of air/volatiles. The condition of the adhesive–adherend interface becomes a decisive factor affecting the initial bond strength and the long-term durability of the bonded joint (Marra, 1992; Pocius, 2002; Pizzi and Mittal, 2003).

Surface preparation

Gluing wood is not normally difficult and it is generally possible to obtain a good bond, provided that adequate surface preparation is undertaken before bonding. The main reasons for preparing the wood surface before bonding are: (1) to produce a close fit between the adherends; (2) to produce a freshly cut or planed surface, free of machine marks and other surface irregularities, from extractives and other contaminants; (3) to produce a mechanically sound surface, without crushing and burnishing it, which would inhibit adhesive wetting and penetration (Pizzi, 1983; Minford, 1991; Adams *et al.* 2004; Richter and Steiger, 2005). With regards to (2) the higher the number of available bond sites the greater the total attraction. It is this attraction which gives freshly machined wood its desirable wettability (Adams and Wake, 1984).

During machining, the disruption of the chemical structure may leave residual charge on the timber surface, also making it very receptive to wetting by the adhesive. This promotes the development of strong adhesion forces, an important requirement for good bonds. Unfortunately, water, gases, microscopic dust and dirt particles, extractives in the wood, are also likely candidates for open bond sites. This, as in the case of many materials, is why wood loses its wettability over time and the surface of the wood becomes inactivated (Mays and Hutchinson, 1992; Davis, 1997; Working Group 1, 2002; Custódio *et al.*, 2008).

Poor wetting is the result of an inactivated wood surface. In this situation, although adhesive penetration may still take place, filling the voids on the wood surface, the adhesive is not molecularly attracted to the wood and as a result a weak bond will occur at the adhesive–wood interface (Adams and Wake, 1984; Marra, 1992).

Wood cells can be damaged by machining, usually involved in the preparation of wood surfaces prior to gluing. A quality planed surface can be obtained with carefully sharpened knives, a shallow depth of cut, and a moderate feed rate. This should produce a clean cut, leaving both earlywood and latewood cells exposed without compression damage, and with a maximum exposure of fresh wood with the least damage to the surface and subsurface cells (Hernandéz, 1994; Department of Agriculture, 1999; Singh *et al.*, 2002).

Besides mechanical damage, thermal damage should also be avoided during any surface preparation. When the wood is overheated or overdried, the chance of an inactivated surface is increased, because heat increases the movement of wood extractives, thus increasing the chance that they will move to the wood surface and attach to open sites. In addition, severe heat can actually alter the chemistry of wood components, destroying available bond sites (Rabiej *et al.*, 1997).

If surface inactivation is detected, several actions may be taken to increase the chance of good bonding. The most effective and traditional method to revitalise wood surfaces for bonding has been through planing. Another way to reactivate the wood surface is to sand the surface to be glued, because sanding raises the critical surface tension of wood, improving its wettability (Custódio *et al.*, 2006c). However, sanding is generally not recommended for preparing wood surfaces for bonding (Murmanis and River, 1983).

A better result is obtained when surface preparation combines an abrasive stage followed by a chemical cleaning stage. If timber is oily, e.g. teak, the surface can be wiped with a volatile solvent such as acetone before applying the adhesive after the solvent has flashed off. If timber is resinous, such as Douglas fir or Oregon pine, a 10% sodium hydroxide solution could be used to wash the face to be glued, followed by a fresh water wash. Wiping with solvent also removes sanding dust. Another approach is to incorporate additives in the adhesive to increase its wettability and penetration into the timber (Minford, 1991; Department of Agriculture, 1999; Šernek *et al.*, 2004).

To prevent surface inactivation in wood, two measures should be taken. First, the wood must be dried, with care taken not to overdry or overheat the wood. Second, the wood should preferably be planed before bonding to remove hydrophobic and chemically active extractives and other physical and chemical contaminants that could interfere with bonding (Davis, 1997; Department of Agriculture, 1999).

Age of surface

Because adhesives bond by surface attachment, the physical and chemical condition of the adherends' surface is extremely important for good joint performance. Immediately after preparation, all wood surfaces undergo an inactivation process. An inactivated wood surface bonds weakly with adhesives, because the inactivation process reduces the ability of an adhesive to properly wet, flow,

penetrate and, in some cases, can influence cure, e.g. phenolic and urea adhesives (Working Group 1, 2002).

Formation of micro-cracks during drying and ageing of wood can reduce the bond strength, since it may lead to an overpenetration of the adhesive and increase consumption of the binder (Back, 1991; Nussbaum, 1993, 1995, 1999; Nussbaum and Sterley, 2002).

To achieve optimal adhesion conditions it is important to control the time-dependence of the inactivation process. It is recommended that no more time than necessary should be allowed to elapse between final surface preparation and bonding. The prepared surfaces should be kept covered with a clean plastic sheet or other relatively inert material to maintain cleanliness prior to the bonding operation. Experimental studies have demonstrated a substantial reduction in wettability during the first 24 h after preparing the surfaces of several wood species. So it is commonly accepted that wood should be surfaced or resurfaced within 24 h before bonding to remove extractives and other physical and chemical contaminants that interfere with bonding (Roffael and Rauch, 1974; Nussbaum, 1995).

Wood species

Wood is a porous, permeable, hygroscopic and orthotropic material of extreme chemical diversity and physical complexity. Because of this, its properties vary between species, between trees within a species, and even within a tree. This variability can lead an adhesive to produce bonded joints that will perform inconsistently and with different levels of performance among the several types of timber species (Working Group 1, 2002).

Wood extractives are complex mixtures of related compounds. Timber can contain a large number of these mixtures that include groups of chemicals such as tannins, anthocyanins, flavones, catechins, kinos, lignans and volatile hydrocarbons. Wood extractives are extremely important because of their often undesirable and unpredictable effect upon adhesive bonding (Marra, 1992).

As a general rule, hardwoods contain more extractives than softwoods. In many species there is a measurable difference between the gluing characteristics of sapwood and heartwood, possibly due to the nature and the amounts of extractives found in the heartwood. The simplest way of minimising the negative effect of the extractives presence is to remove them before bonding (Roffael and Rauch, 1974; Back, 1987; Marra, 1992; Belfas and Grooves, 1993; Balfas, 1994; Department of Agriculture, 1999; Working Group 1, 2002; Pizzi and Mittal, 2003; TRADA, 2007).

Treated wood

Depending on the species and the end-use, timber can be treated with chemicals to enhance its performance against biological agents, fire and weather. Wood can be protected from the attack of decay fungi, harmful insects, or marine borers by applying chemical preservatives. Timber protection against fire can be achieved through its impregnation or painting with fire retardants. All these treatments should be considered as contaminants, as far as adhesion is concerned. Types of surface chemical treatments, adhesives, conditions of joint assembly and adhesive cure can have varied and strong effects on the strength and durability of bonds. Certain combinations of these factors can lead to excellent bonds, despite the interference from chemical treatments (Miroy *et al.*, 1995; Davis, 1997; Department of Agriculture, 1999).

Several studies (Vick and Groot, 1990; Vick and Kuster, 1992; Dimri and Shukla, 1993; Vick and Christiansen, 1993, 1997; Vick *et al.*, 1993, 1998; Manbeck and Shaffer, 1995; Vick, 1995, 1997; Sellers and Miller, 1997; Kilmer and Blankenhorn, 1998; Tascioglu *et al.*, 2003; Herzog and Goodell, 2004; Lisperguer and Becker, 2005) concerning the influence of different preservative treatments on the strength and durability of bonds were conducted on joints bonded with traditional adhesives, such as phenol-resorcinol-formaldehyde (PRF), resorcinol-formaldehyde (RF), melamine-formaldehyde (MF), melamine-urea-formaldehyde (MUF), emulsion-isocyanate (EPI). However, studies about the preservative treatments, effects on bonds using contemporary adhesives such as epoxies are practically non-existent.

Experiments have shown that, in spite of the negative effects that chemical preservatives bring to adhesion, many types of preservative-treated timber can still be successfully bonded with specially formulated adhesives and by closely followed specific procedures (Working Group 1, 2002; Tascioglu *et al.*, 2003).

Fire retardants increase the hygroscopicity of the timber and can affect adhesion to the timber surface. The most common fire-retardant chemicals used for wood are inorganic salts based on phosphorous, nitrogen and boron. The acidity of these chemicals can hinder the cure of neutral or alkaline curing adhesives such as RF or PRF. However, strong and durable bonds can be made to certain fire retardant-treated woods by washing the surfaces to be bonded with certain alkaline aqueous solutions, e.g. 10% caustic solution, followed by a drying stage before bonding (Davis, 1997; Department of Agriculture, 1999).

Stress

The main purpose of a structural adhesive joint, or any structural joint for that matter, is to transfer load from one component to another. In the case of adhesive joints, this frequently translates into a connection between dissimilar materials that have very different material properties. Hence, the more traditional timber joints, such as the scarf joint or tenon-type joints, have been ignored. In order to characterise the mechanical behaviour of an adhesive joint these material properties together with the means in which the load is transferred, i.e. joint geometry,

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need to be thoroughly understood. This issue is discussed in detail by Broughton and Hutchinson (2001c, 2001d).

Influence of material stiffness on stress

In transferring load, the stiffness of the various materials involved is a key factor in the development and magnitude of resultant stress concentrations. In the particular case of timber connections bonded on site, this often involves metallic or FRP rods, or plates, bonded into either new and/or aged timber. Thus, bonded joint configurations consist of a combination of dissimilar materials exhibiting a large property mismatch between the rod and other materials. Hence, the resultant stress peaks arising from the externally applied load are at a minimum when stiff adherends (e.g. steel or FRPs) and/or flexible adhesives (e.g. some acrylics and polyurethanes) are involved. Conversely, they are at a maximum when more flexible adherends (e.g. timber) or rigid adhesives (e.g. stiff epoxies) are present.

Influence of joint geometry on stress

The geometry of the joint itself is a key parameter in the evaluation of bond performance. Generally joints, whether used for adhesive characterisation or durability testing, fall into two groups, where: (1) the test configuration closely matches that of the real connection; and (2) the joint geometry is very different from that of the real connection design (in this case only certain aspects are focused upon). In the latter case (2), adhesion is often the main focus and is assessed by employing test configurations that highly stress the bonded interfaces in peel (e.g. 90° or 180° peel tests), cleavage (fracture tests) or direct tension (e.g. butt test, pull-off test). In the former case (1), lap shear or co-axial bonded joints are often preferred, whereby the loading is supposedly acting in shear, as in the case of the on-site connection.

It is common for designers and engineers to prefer joint configurations from Group 1, as they provide joint strength data. On the other hand, adhesive technologists and scientists prefer those from Group 2, as they tend to separate adhesion from joint strength, which is inherently related to the joint geometry. Indeed, adhesion is often discussed in relation to the strength of joints, but the force required to fracture a joint is resisted by a complex interaction of internally generated stresses and strains. Attempts to use joint strengths as a measure of adhesion can therefore be extremely misleading.

Various types of joints have been developed and the most common of these are described as:

- single lap joint: compression or tension (Adams *et al.*, 1997)
- co-axial joint (Broughton and Gardelle, 2005; Richter and Cruz, 2007), with characterisation of bonded-in rods described in (CEN/TC 193/SCI/WG11, 2003; Custódio *et al.*, 2007)
- peel joint (Adams *et al.*, 1997)
- modified butt joint (CEN/TC 193/SCI/WG11, 2003)
- fracture joint (Gustafsson, 1987; Gustafsson and Serrano, 2000; Broughton and Gardelle, 2005).

Joint selection for the assessment of bond performance

The majority of test joint configurations noted above develop bondline stresses that are far from uniform. Failure loads are therefore related to stress concentrations at the ends (or perimeter) of the joint and the toughness of the adhesive. Notably, if the bonded area is sufficient to enable stress redistributions within the adhesive layer, the peak stress concentrations can be further affected due to changes in the adhesive material properties as a result of environmental exposure, initially giving apparent increases in joint 'strength' and 'toughness'.

Thus the size of test joints, as well as their geometrical configuration and the type and nature of the adherends involved, affects bond performance. It generally takes longer for environmental effects to become apparent in large joints unless initial adhesion is poor. This is because moisture-induced problems are related to diffusion through the adhesive layer from the end, or perimeter, of the joint.

If the adherends are permeable to water, as is the case with timber, then relatively large bond areas can be utilised for testing as the time taken to saturate the bondline is somewhat shorter and environmental effects will become apparent sooner. In the case of impermeable adherends, test joints with small bonded areas, or with large bondline perimeters compared to their bonded areas, are generally preferred for durability testing. (Kinloch, 1987; Annon, 1994; Beevers and Bowditch, 1995; Bond *et al.*, 1996; Taylor and Kinloch, 1998; Weitzenböck *et al.*, 1998; Fitton and Broughton, 2005).

Summary

Essentially, whatever the joint configuration, a bonded joint represents a layered system comprising different materials and interfaces, all of which respond in different ways to an externally applied load and/or change in environmental conditions. As a result, a number of competing mechanisms are taking place during service life. There is clearly a need for the successful prediction of long-term performance and suitable accelerated ageing tests throughout the many fields and industries that involve adhesively bonded structures. It is essential that future research effort for bonded timber connections should encompass the lessons from previous work and focus upon joint combinations and configurations that will provide realistic data for the specific materials under consideration. Current strategies for long-term durability testing and accelerated ageing should therefore encompass the following factors:

- Changes due to environmental exposure tend to be more detectable in smallish joints (small bond area).
- No single joint configuration and test procedure can provide all of the information necessary to provide a definitive measure of bond performance.
- Several procedures are required to separate environmental effects on adhesion, the adhesive and the adherend.
- Temperature is a vital component of the durability assessment procedure of adhesive joints but excessive temperature durations are not generally considered realistic and should be used with special caution for accelerated durability evaluation.
- Stress is a crucial component for the evaluation of durability but excessive levels of stress should be avoided and should be used with special caution for accelerated durability evaluation.
- Moisture is a critical and especially harsh component and as such is a necessity for any durability testing of adhesives. High levels of moisture can affect bonds and, in this case, it is generally accepted that excessive levels can be used to accelerate degradation.
- Many joint configurations have been employed, see **Figure 6** for joint types without particular reference to substrate limitations or exposure environments, and it is clear that induced peel or cleavage loading (Group 2 type) provides the more discriminating joint configurations to assess bond performance, but relating this quantitatively to long-term service life prediction is difficult without also assessing joints similar to that employed in practice, i.e. Group 1 type.

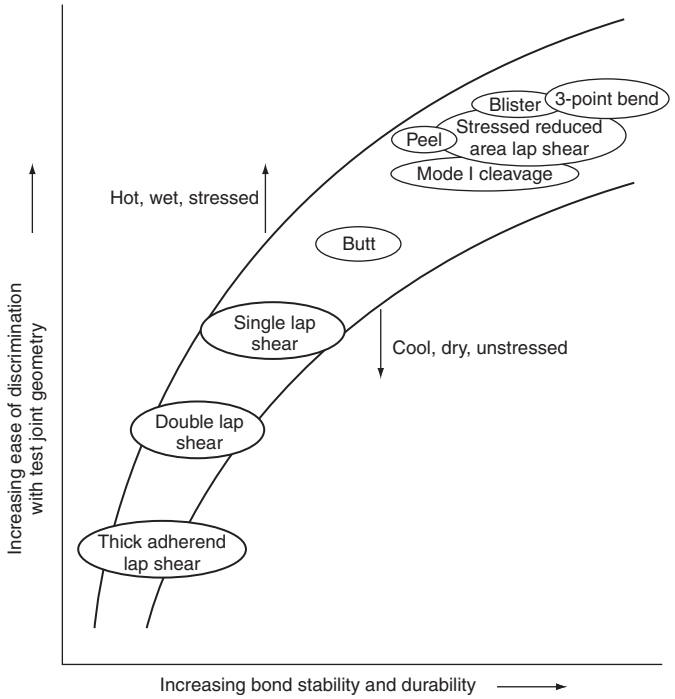

Figure 6 General suitability of current test specimen geometries to investigate interfacial bond stability and environmental durability (adapted from Broughton and Hutchinson, 2001)

Testing and performance

Chemical and mechanical testing of polymers

Because an adhesive should be able to maintain the integrity of the bonded assembly under the expected conditions of service for the life of the structure, it is important to understand the properties of an adhesive that allow it to perform this function.

Fundamental tests can be used to characterise the adhesive. For instance, the molecular structure of adhesive polymers are chemically characterised spectroscopically by nuclear magnetic resonance (NMR), either in the liquid or solid state, electron spectroscopy for chemical analysis (ESCA), Fourier transform infrared spectroscopy (FTIR), etc. Molecular-size distributions of polymers are determined by gel permeation chromatography (GPC). Rates of chemical reaction are studied by differential scanning calorimetry (DSC). It is also important to understand the viscoelastic behaviour of the adhesive under other conditions, for example, since for wood adhesives, the two most important changes in conditions are changes in temperature and moisture, their effect on the adhesive mechanical and rheological properties can be assessed by dynamic mechanical analysis (DMA), tensile tests, shear tests, etc. The thermal stability of adhesives can be studied also by thermogravimetric analysis (TG).

Much can be learned from measurements of chemical, mechanical and rheological properties of adhesives as bulk materials. Nevertheless, for a bonded assembly, the overall properties are more difficult to predict because less is known about the properties of the interphase regions compared to the bulk properties of adhesives and adherends. In addition, the differences in mechanical properties of the wood, adhesive, and interphase regions imply that stress concentrations are likely to occur in the zone of greatest change, i.e. the interphases zone. With environmental exposure, the interphase has to accommodate the large dimensional changes between the wood and the adhesive, so it is important to remember that the internal forces can be as significant as the applied forces. This is imperative to test the performance of the bonded joint through bonded assemblies prepared with specific adhesives and adherends, and tested under specific loading modes, environmental conditions, and strain rates that will match as closely as possible those experienced in service by the joint in the structural bonded joint.

Mechanical testing of bonded assemblies

Many factors that can affect the strength of the joint can be measured by mechanical testing. Sometimes it is difficult to separate the variables that control joint strength, but at least through testing, a quantitative approximation can be made. These tests can be conducted for a variety of reasons: (1) to compare strengths of various bonding systems to

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choose suitable materials or processes; (2) to monitor the quality of production materials to assure they have not changed since the last time they were verified for use in the bonding process; (3) to compare and confirm the effectiveness of a bonding system, using different adhesives, curing schedules, surface treatments, surface cleaning, etc.; or (4) to investigate parameters or process variables that may lead to measured differences in the performance of the bond.

A number of standard tests for bonded assemblies have been specified by professional organisations, such as the American Society for Testing and Materials (ASTM), the European Committee for Standardisation (CEN), International Organisation for Standardisation (ISO), etc. However, standard test results cannot be readily translated into specific strength values for an actual production joint. The main reasons for the differences between standard test results and the results from an actual joint in service are due to several factors, namely: (1) actual joints generally have a complex geometry that can be significantly different from the standard test specimen geometry; (2) the mode and application of stress are usually different and more complex in practice; (3) environmental ageing is usually less severe in service, and the effects of the actual service environment are generally more complex (e.g. when in service, the joint may be simultaneously exposed to cyclic stress, cyclic temperature and humid environments); (4) laboratory test specimens are usually made in 'controlled' environments; (5) the number of specimens tested is limited due to time constrains; (6) the materials (adhesives and adherends) tested in the laboratory are, usually, very uniform and from the same batches, due to the need to reduce variability; and (7) even with laboratory specimens, a full design-of-experiment statistical process is difficult to achieve because of the many production variables that can affect joint strength (Kinloch, 1983; Marra, 1992; Pizzi and Mittel, 2003).

Therefore, the most reliable test would be measuring the strength of an actual assembly under real operating conditions. Unfortunately, such tests are often expensive or impractical. Thus, the next best and feasible method is to measure the strength of an actual assembly under simulated operating conditions that are as close as possible to those existing in service. The usefulness and limitations of each standard testing method should be understood clearly before selecting a specific test program, so that the user can choose the test geometry, procedure, and methodology that best serve the application. In some cases, it may be even necessary to develop a new test method to meet the characteristics of a specific application.

Long-term performance tests

For an adhesive bond to be useful, it not only must withstand the mechanical forces that are acting upon it, but it must also resist the elements to which it is exposed during service. Thus, one of the most important characteristics of an adhesive joint is its durability. ASTM D907 defines durability 'as related to adhesive joints, the endurance of joint strength relative to the required service conditions'. The effect of simultaneous exposure to both mechanical stress and a service environment is often more severe than the sum of each factor taken separately. Moreover, each adhesive responds differently to those factors. Consequently, it is understandable why no laboratory exposure treatment has yet or will be developed that is able to predict the behaviour of even a single product in more than one limited environment. Thus, it is usually necessary to test bonded joints, both in the laboratory and in the field, and under conditions as close to the actual service environment as possible. The long-term performance test should allow determining the ultimate strength and durability of the adhesive and the bonded product. The parameters that will likely affect the durability of a bonded joint are: (1) maximum stress level; (2) average constant stress level; (3) nature and type of environment; (4) cyclic effects of stress and environment (rate and period); and (5) duration of exposure.

Because structural wood bonded joints are made to last several decades, it is desirable to know the rate at which an adhesive bond will lose strength due to environmental factors in service. Since the strength values determined by short-term tests do not always give an adequate indication of an adhesive's performance during continuous exposure to the service environment, laboratory accelerated artificial weathering tests are then used to estimate the long-term performance. These tests are accomplished by exposing a stressed or unstressed joint to simulated operating conditions, and the durability is measured by the loss or rate of loss of a property, usually a mechanical property, against the time or number of treatment cycles of an artificial accelerated or natural exposure test. The time- or cycle-dependent changes may be chemical or mechanical. Exposure is typically to elevated temperature, humidity, water, or various chemical solutions that are representative of the service conditions, and then the results are extrapolated to actual conditions. However, often too many interdependent variables and modes of strength degradation are in operation, and a reliable estimate of life using simple extrapolation techniques cannot be achieved.

The most common problem with wood durability is the adhesive's inability to withstand the swelling and shrinking of wood with moisture changes. The swelling of wood can subject the bond to Mode I, II, or III types of loading depending on the joint design. The normal force of mode I is perpendicular to the bond in the direction in which adhesives are weakest. On the other hand, the shearing force of mode II is the direction in which the adhesive is the strongest. The less common torsion forces of mode III

are an intermediate test of adhesive strength. Another factor that has to be considered is that accelerated tests involve rapid wetting and drying of the wood. These changes can be so fast that the wood does not have chance to stress relax during the test; thus, artificially high stresses may be created that would not otherwise be observed in normal use. This is why stressed ageing tests are important and why they more accurately simulate service conditions than do simple tests where the specimens are merely placed in a test environment. The type of stress and environment should be selected to be a close approximation of real-life conditions, so that the acceleration does not cause reactions or mechanisms within the materials or bond-line that would not actually be present in the intended real environment. Therefore, testing must be combined with understanding of the principles of adhesion and the many processes that are involved (Kinloch, 1983; Marra, 1992).

Health and safety

All works involving adhesives should be executed in accordance with health and safety regulations, local regulations and, if used on site, according to the contractors' presubmitted Health and Safety Plan. The use of adhesive requires special handling and record keeping. All materials require a Material Safety Data Sheet (MSDS). This MSDS supplies information about the make-up of the material, identification of hazardous ingredients, fire and explosion data, health hazard information, reactivity data, disposal, spill and leak procedures, and special protection information. All handling and disposal of hazardous materials must be properly controlled, monitored, documented and must follow the most appropriate current regulations (Richter and Cruz, 2007). Some of the Health and Safety regulations which need to be taken in consideration when using adhesives in construction are listed below:

- The Health and Safety at Work Regulations 1999
- The Construction (Design and management) Regulations 1996
- The Construction (Health, Safety and Welfare) Regulations 1996.

Quality control

An effective quality assurance programme should be conducted to assure satisfactory end-product strength and durability. A typical quality assurance programme consists of three parts: (1) establishing limits on bonding process factors that will ensure acceptable joints and product; (2) monitoring the production processes and quality of the bond; and (3) detecting unacceptable joints and product, determining the cause, and correcting the problem. Materials and material handling should fulfil the requirements in the applicable specifications, including proper storage and the compliance to shelf-life stipulations. After completion, the quality of the finished product should be evaluated by the responsible quality assurance personnel (Marra, 1992; Richter and Cruz, 2007).

Future developments

Wood is a traditional construction material still widely used today. The sustained use of wood in construction is due not only to traditional reasons but also to the excellent characteristics of this material. Moreover, it is convenient and easy to process and, due to sustainable forestry, also available in sufficient quantities and good qualities. In the past decades, the development of new wood products has significantly expanded the material's range of applications in the construction industry. Due to environmental concerns, we will continue to see an increase in the use of bonded timber products. Also, new applications for bonded wood products and greener adhesives will appear. For instance, timber-concrete-composites with an adhesive connection, prestressed timber and timber–concrete decks, on-site repair and strengthening of existing timber structures, green gluing, wood welding, thermomechanical densification of timber, etc., are relatively new and emerging research fields.

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Further reading

This section lists technical literature and organisations that provide specialist advice on adhesive technology.

2005 Wood Design Package that includes: National Design Specification (NDS) for Wood Construction with NDS Commentary; and NDS Supplement – Design Values for Wood Construction Special Design Provisions for Wind and Seismic (Wind & Seismic), with Commentary ASD/LRFD Manual for Engineered Wood Construction, 2005 edition.
Adhesive Bonding: Science, Technology and Applications, R. D. Adams (Ed.), Woodhead Publishing Limited, 2005.
Building Research Establishment Ltd, BRE – provides a complete range of consultancy, testing and commissioned research services covering all aspects of the built environment, and associated industries.
Institute of Wood Science, IWSc – aims to promote and encourage a better understanding of timber, wood-based materials and associated timber processes. In conjunction with a number of colleges and training agencies in the UK and Ireland the Institute organises courses in wood technology and utilisation; and awards appropriate qualifications (http://www.iwsc.org.uk/).
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Resin Repairs to Timber Structures: Vol. 1 Guidance and Selection. Report 3/2000, 2000.
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Rotafix Ltd – has been in operation since 1974 and the company is specialised in repair and restoration techniques for all types of structures. Since 1985 the company has been producing its own specialist adhesives and grouts for use in the construction industry. The company offers specialist training, in particular for the restoration of historic building fabric including timber. Rotafix offers on-site surveys, strength grading of timber and laboratory analysis (http://www.rotafix.co.uk/).

The Adhesives Design Toolkit (http://www.adhesivestoolkit.com/Introduction.xtp) – is a web-based resource that provides a framework for the collation and organisation of relevant material related to adhesives and adhesive bonding. It provides a systematic approach to enable users to weigh up key factors involved in the design of an adhesive joint.

The Timber Research and Development Association, TRADA – is a membership-based organisations and internationally recognised centre of excellence on the specification and use of timber and wood products.

TRADA provides to members information on timber and wood products to enable them to maximise the benefits that timber can provide. It provides a wide range of commercial and training services to TRADA members and to the timber and construction industries (http://www.trada.co.uk/).

TWI World Centre for Materials Joining Technology Ltd, which holds The UK's Centre for Adhesive Technology (http://www.twi.co.uk) – advises companies on: applications where adhesives can be used in place of traditional joining methods; selection of adhesives for specific materials; joint design for optimum quality and productivity; bonding procedures; and quality assurance.

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Chapter 64

Hazards, specification and service life performance

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The fact that wood is a complex biological material means that it is potentially vulnerable to certain types of hazards, which if present, can lead to degradation of structural performance and a corresponding reduction in effective service life. In order to avoid this phenomenon, it is important to identify potential hazards and understand how to specify timber appropriately to ensure that service life expectations are met.

Introduction

In this chapter, the specification of timber in relationship to hazards and the resulting service life performance is presented and discussed. Hazards, as for all structures, can be many, but generally are considered to be of mechanical, biological, physical or chemical nature, the first two being the most important for timber structures during normal service life. Fire safety is considered as a special case of a physical load. In addition, the natural resistance against chemicals makes wood a very good alternative for chemical storage facilities. The choice of wood species and the possible preservative treatment is discussed in relation to its natural durability and the end-use in structures. Structural detailing is highlighted as the most important design aspect to obtain long service life, but apart from good structural detailing, the right choice of wood species and/or treatment and a correct assessment of the climate to which the structure will be exposed are essential. Future changes need to be anticipated as well, for instance an increase in traffic loads on bridge and decks or changing climate conditions as a result of changing human activities inside buildings.

Hazard types

The behaviour of timber structures depends to some extent on the hazards against which they are designed. Hazards are distinguished in four different categories: mechanical loads, biological, physical and chemical hazards.

Mechanical loads

For the structural engineer, mechanical loads are considered the most important and the structure will be designed against deadweight loads and a number of variable (live) loads such as imposed loads, snow, wind and earthquakes. Generally, the level of these loads is specified in design codes and standards, such as Eurocode 0 and Eurcocode 1. Whereas, for imposed loads and snow and wind loads the load-carrying capacity of members and joints is essential, in earthquake zones the keyword is energy dissipation. Good energy dissipation can be obtained by plastic deformation in joints and connections; friction is also a key contributor. Designing with cross-laminated timber panels allows also for a completely elastic behaviour of multi-storey buildings during earthquakes (Ceccotti *et al*., 2006). Special impact loads may also need to be considered. Such loads may occur for instance for timber roadside safety systems, bridge railing, poles, mooring systems in terms of impact loads for poles, guard rails and bridge railings.

Biological hazards

The combination of biological 'loads' (resulting from hazards) with mechanical loads can be considered the most important load events when considering service life performance. Biological attack may lead to structural degradation and can be identified in three subcategories.

Fungi

Wood destroying fungi are generally divided into brown-rot, white-rot and soft-rot. Brown rot and white rot draw their names from the colour of the residuals after attack. Soft-rot makes the timber weak because of the cavities they create. The five essential factors necessary for the growth and presence of active fungi are the right combination of food (timber), oxygen, moisture, temperature and acidity (pH). In particular, moisture contents in the range between 20–40% are quite conducive to fugal attack and should therefore be avoided as far as possible.

Apart from the wood-destroying fungi, some fungi may just discolour the wood surface. Mould and blue stain fungi are included in this group. These fungi are not considered a hazard for structural safety, but may lead to an unappealing surface quality of the wood and degradation (minicracks, deterioration of the coating, and consequently a visual degradation).

Bacterial degradation

Bacterial degradation is the slowest process and for the most part occurs when timber is used in in-ground applications. When fungal decay is present, there will often be a small amount of bacterial decay as well. However, bacterial decay only becomes important when timber is used in the ground and below the groundwater level. In general, the growth of bacterial decay is quite limited at a maximum of around 0.2 mm/year. Piles made of pine are more susceptible to bacterial decay than other species and therefore the use of such piles is generally prohibited without preservative treatment. Other softwood piles may be used below groundwater level without any further treatment, because of the low oxygen content.

Insects

Insects that destroy the timber are divided into two subgroups: beetles and termites. Beetles generally do not destroy the timber themselves but their larvae do. The insects lay their eggs in the timber or on the surface. After the eggs hatch the larvae penetrate the timber and create holes and cavities that effectively result in localised destruction of the wood. Evidence of attack is usually indicated by the presence of wood dust on the surface near the beetle holes.

Termites

Termites are considered to be a high-risk hazard to timber in a number of regions around the world – predominantly in warmer temperate and tropical climates. Even though termites do not move around very quickly, once infestation has occurred, termite attack can lead to serious structural degradation in a relatively short period of time.

Marine borers

Two types of marine borer are common almost worldwide: shipworm, **Figure 1** (*Teredo* spp./*Bankia* spp.) and gribble, **Figure 2** (*Limnoria* spp.). The risk of attack from marine borers is influenced by water temperature (minimal 5°C) and salt level (minimal 7%). In addition, certain types of localised pollution have the potential to contribute to creating an environment that is favourable to marine borers, although, generally speaking, in polluted water areas there is a lower risk of infestation than in pristine water conditions. However, current efforts to improve water quality in polluted rivers and estuaries have been recognised as a factor that has contributed to increased risk of marine borer attack. The typical decay pattern of a shipworm is difficult to discover, since the entrance holes are small, whereas the member can be completely ruined internally (see **Figure 3**). The decay pattern of gribble attack is easier to recognise. Generally, the wood is eaten from the outside to the inside and the piles are cut in two, see **Figure 2**.

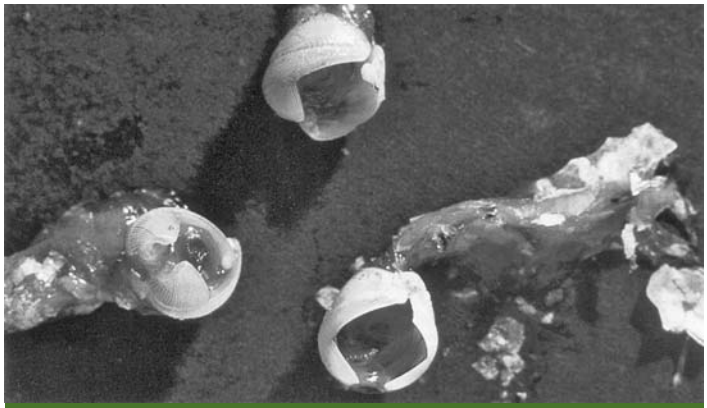

Figure 1 Shipworm (Rijkswaterstaat, NL)

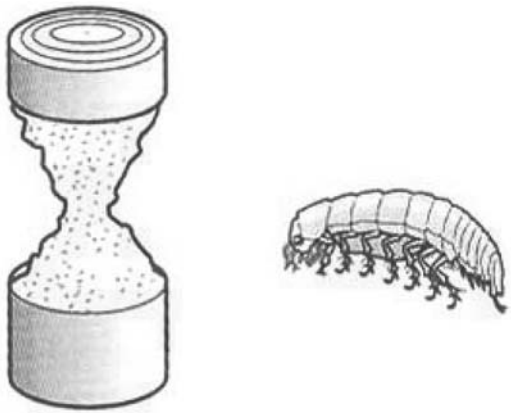

Figure 2 Decay pattern of gribble

Physical hazards

Moisture and temperature

The most important physical hazards are moisture and temperature, with temperatures under fire conditions being the most extreme. Normally, temperature or temperature variations do not cause serious problems to structural timber. Expansion due to temperature is considered to be counteracted by shrinking due to a decrease in moisture content and, as such, there is generally no need to allow for expansion joints in the design of a structure. However, at elevated temperatures the hemicellulose substances in wood soften and weaken. Consequently, the use of timber structures is not recommended when the temperature may rise over 60°C for prolonged periods as, if this occurs, increased creep deformations may be expected.

Glued laminated timber structures may suffer from high temperatures in certain types of structures. In centrally heated indoor climates small cracks can develop or local delamination can occur in members exposed to heated dry air that has accumulated due to the heating. In such situations it is quite common to find that wood moisture contents have dried below 8%.

Another possible hazard that can lead to high temperature in timber occurs when dark surfaces of structural

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Figure 3 Cross-cut of a pile of shipworm decay

components, either when they have been painted or treated for durability, are directly exposed to outdoor climate influences. An example is shown in **Figure 4**, where small cracks and local delamination have occurred at the support hinges of a three-hinged portal frame. In this instance, the supports are fully exposed to direct climatic influences of moisture and temperature as well as direct rain and sun exposure. To avoid such deterioration, timber should be protected as far as possible from direct exposure to rain and/or sun. Furthermore, large timber joints need to be designed in such a way that shrinking and swelling can occur without introducing tensile stresses perpendicular to the grain.

Fire

The most serious physical hazard for structural and occupant safety is fire. Wood burns, develops flames and smoke when burning, and consequently can also contribute to the spread of fire. Modern fire safety concepts are based on three concepts: (1). The most important thing is to try to extinguish a fire (e.g. by the use of active sprinkler systems) and prevent the possibility of 'flash-over' when it can develop into a full fire. If this is not possible, structural

Figure 4 Delamination and cracks of glued laminated timber at the supports

safety and integrity must be guaranteed by (2) containment in order to avoid the fire from spreading outside the room or compartment and (3) fire rating critical elements to ensure that collapse will not occur within a specified period to enable occupants to escape and fire fighting crews to enter and leave the building safely.

Fire-retardant treatments can improve the performance of wood and wood products in case of fire. While the wood burn rate is normally not decreased, their main contribution is the reduction of ignitability, flame spread and release of heat. Consequently, such treatments are very useful in non-structural products, both in the internal parts of a structure as well as for outside claddings. Fire-retardant materials can be applied by coatings, by pressure impregnation or during manufacture of wood-based products.

Above 250°C wood starts to burn and major constituent components in the structure of wood, such as cellulose, start to disintegrate.

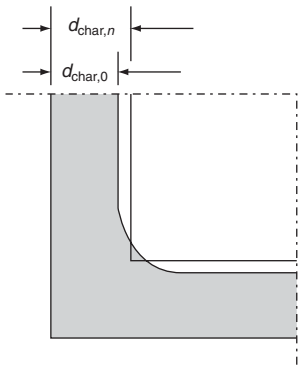

Figure 5 Charring depth $d_{char,0}$ for one-dimensional charring and notional charring depth $d_{char,n}$ according to Eurocode 5

Design against fire can be based in two principles, which may be combined. The first principle takes advantage of the effect that timber in fire conditions creates an isolating char layer. This char layer results in a constant burning of the timber over time. For most softwood, the burn (or char) rate is about 0.7 mm/minute. Consequently, the designer can estimate the residual cross-section after 30, 60 or 90 min, whichever is required. Denser hardwoods have lower burning rates and consequently retain their load-carrying capacity for a longer time.

The background of fire safety calculations is based on the charring rate of timber, which is assumed to be linear with time. In most cases the notional charring rate $d_{\mathrm{char},n}$ is used, which takes into account the effect of the roundings in rectangular beams. This is a safe approximation and makes it easy to calculate member cross-section properties. The charring and notional charring depth can be calculated as:

$$d_{\mathrm{char},0} = \beta_0 \, t$$
$$d_{\mathrm{char},n} = \beta_n \, t \qquad (1)$$

The basic charring rates for different materials are given in **Table 1**. For other species, the basic charring rates may be modified as:

$$\beta_{0/n,\rho,t} = \beta_{0/n} \, k_\rho \, k_h \qquad (2)$$

$$k_\rho = \sqrt{\frac{450}{\rho_k}}$$
$$k_h = \sqrt{\frac{20}{h_p}} \qquad (3)$$

The second principle applied to design against fire is the use of protective layers. This is normally done in timber-framed housing, where gypsum boards are used to protect the timber from burning, at least during the initial stages of the fire.

UV degradation is a relatively small 'risk' in terms of degradation and structural safety. Basically, UV radiation degrades the timber surface up to a depth of maximal 0.5 mm. Discoloration is the result and the wood surface slowly turns grey.

Chemical hazards

Timber is ideally suited for structural applications in the chemical industry. Examples are storage and production facilities in the industry or for the storage of de-icing salts for roads. Chemically 'polluted' atmospheres may also be present in swimming pools, spas and in coastal regions where the air contains salt. Timber by nature is slightly acidic to acidic, with pH values between 5.5 and 2.5. In the case of chemical actions, the degradation or 'corrosion' of the timber always starts from the outside of the members. The cellulose is generally resistant to most chemical actions, also due to its crystalline nature, but the chemicals may attack the hemicelluloses and the lignin. In environments where the acidity ranges between pH 3 to 10, no particular influence on the resistance of timber is found. When exposed to pH values less than 2, degradation of all wood substances – cellulose, hemicelluloses and lignin – will occur. At the other end of the spectrum, pH values between 11 and 14 (ammonia, caustic soda) will cause wood to degrade (similar to the de-lignification process used to produce paper products), with softwoods showing a better resistance than hardwoods. Gases such as ammonia generally do not influence mechanical resistance, indicating the good applicability of timber for agricultural applications.

While chemical hazards may affect timber, often the most severe impact will be on steel fasteners that are subject to corrosion. This may be caused either by the climatic conditions, or by the extractives present in the wood members. This effect depends to a large extent on the wood species used, but species known for their corrosive properties are oak, western red cedar, Douglas fir and chestnut. In these species in particular, the tannins are responsible for the corrosive behaviour. Apart from the species, chemical treatments may also affect the fasteners. Both preservative treatments (salts generally containing copper) as well as fire retardants (generally acid) may corrode steel fasteners. Apart from the fact that the fasteners may lose their functionality, the corrosion may also result in stains on the wood surface. **Table 2** gives guidance for steel fasteners as a function of climate.

	β_0: mm/min	β_n: mm/min
(a) Softwood and beech		
Glued laminated timber with a characteristic density of $\geqslant$290 kg/m^3	0.65	0.7
Solid timber with a characteristic density of $\geqslant$290 kg/m^3	0.65	0.8
(b) Hardwood		
Solid or glued laminated hardwood with a characteristic density of 290 kg/m^3	0.65	0.7
Solid or glued laminated hardwood with a characteristic density of $\geqslant$450 kg/m^3	0.50	0.55
(c) LVL with a characteristic density of $\geqslant$480 kg/m^3	0.65	0.7
(d) Panels		
Wood panelling	0.9[a]	–
Plywood	1.0[a]	–
Wood-based panels other than plywood	0.9[a]	–

[a] The values apply to a characteristic density of 450 kg/m^3 and a panel thickness of 20 mm; see 3.4.2(9) for other thicknesses and densities.

Table 1 Charring rates for wood and wood products (courtesy of Rijkwaterstaat, NL)

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Fastener	Service class[b]		
	1	2	3
Nails and screws with $d \leqslant 4$ mm	None	Fe/Zn 12c[a]	Fe/Zn 25c[a]
Bolts, dowels, nails and screws with $d > 4$ mm	None	None	Fe/Zn 25c[a]
Staples	Fe/Zn 12c[a]	Fe/Zn 12c[a]	Stainless steel
Punched metal plate fasteners and steel plates up to 3 mm thickness	Fe/Zn 12c[a]	Fe/Zn 12c[a]	Stainless steel
Steel plates from 3 mm up to 5 mm in thickness	None	Fe/Zn 12c[a]	Fe/Zn 25c[a]
Steel plates over 5 mm thickness	None	None	Fe/Zn 25c[a]

[a] If hot-dip zinc coating is used, Fe/Zn 12c should be replaced by Z275 and Fe/Zn 25c by Z350 in accordance with EN 10147.

[b] Service Classes 1, 2 and 3 coincide with hazard classes 1, 2 and 3/4 respectively from Table xxx. For especially corrosive conditions consideration should be given to heavier hot-dip coatings or stainless steel, for instance in coastal regions, saltwater applications and treated timber.

Table 2 Examples of minimum specifications for material protection against corrosion for fasteners (related to ISO 2081)

Timber specification

Timber has to be selected depending on the application and required performance, the hazards to which it will be exposed and specific local circumstances. Traditionally, with regard to inherent material properties that resist degradation, timber is classified into durability classes. Normally, four to five durability classes (depending on the location, Europe has five, Australia has four) are distinguished from very durable to non-durable. This system is used to classify the natural resistance of the heartwood of the species against fungi. The sapwood of almost all timber species is never durable, so depending on the type of tree, sawn timber products may still have some sapwood and as such, for outdoor applications, the amount of allowable sapwood will generally be restricted. Depending on the final application, some sapwood may be accepted as long as the timber is exposed to less severe conditions than ground/water contact.

The natural durability is a classification obtained from a so called 'graveyard test' (EN 460 Natural durability of solid wood – Guide to the durability requirements for wood to be used in hazard classes). Small poles of $50 \times 50\,\text{mm}^2$ are placed in the ground and the time (years) is determined until the pole is easily broken by a small impact load. The results of such tests are dependent on the local environment and terrain in which the test is performed, but generally the classification tends to accord with the natural durability ratings presented in **Table 3**. However, due to variations in local conditions (soil, temperature, humidity, impact test, etc.) it should be noted that the classification given below is indicative only and local results of similar tests may differ.

Where timber members are used in saltwater there is the risk of marine borers. This requires special attention because not all species have good durability in a marine environment. Resistance of species against marine borers has been given its own classification, from durable (D), moderately durable (M) and susceptible/slightly durable (S). The following list of species is given which has proven itself in saltwater applications.

If species are used that are not naturally durable for the intended use, the timber will normally require to be chemically treated with preservatives. Wherever timber can reach moisture contents above 20% for prolonged periods (i.e. the EMC is over 20%), it will normally be necessary to prevent biological degradation. Examples where this commonly occurs are marine environments, electricity/utility poles (particularly below ground level), sills and, depending on location and climate, timber in façades. Traditionally, three different preservative classes were distinguished, but recently wax-type treatments have become popular, mainly for environmental reasons.

1. Tar oil preservatives are among the oldest chemical treatments, whilst creosote and creosote/coal tar mixtures are often used for timber bridges, sleepers, electricity poles and many other outdoor applications where good durability and reliability is required.
2. Waterborne preservatives are mineral salt solutions. They are normally referred to as CCA-treatments but a wide range of products are available, with CC – chromated copper – as a common substance. CCA is chromated copper arsenate, but arsenic has been banned in many areas from most applications because of its potential risk to both environment and health. Other combinations are CCB (CC-borate) and CCFK (chromated copper silica fluoride). These products are applied using a vacuum-pressure method to obtain deep penetration into the timber.
3. Organic solvent preservatives (e.g. LOSP) are generally applied on the surface and have less retention than tar, oil or waterborne preservatives, but have good functionality in building components as windows and cladding.

Normally, the chemicals are fixed inside the wood using a pressure treatment process. The timber is placed into a large tank (as seen in **Figure 6**) and a sequence of

Class 1 Very durable (>25 years)	Class 2 Durable 15–25 years	Class 3 Moderately durable 10–15 years	Class 4 Slightly durable 5–10 years	Class 5 Not durable <5 years
Softwoods				
	Alerce	Agathis/Kauri	Sitka spruce	
	Californian redwood	Pine, American	European pine	
	Western red cedar	Larch	Hemlock	
	White cedar	Oregon pine	Parana pine	
			Sitka spruce	
			Weymouth	
Hardwoods				
Afrormosia	Bossé	Dabéma	Abachi/Obeche	Ash
Afzelia/Doussié	Cedar	Danta/Kotibé	Abura	Alder
Azobé	Chestnut, sweet	Dibétou	Avodiré	Balsawood
Basralocus	Framiré	Karri	Elm	Birch
Bilinga	Louro vermelho	Keruing	Hickory	Beech
Demerara greenheart	Mahogany, American	Khaya	Limba	Ilomba
Ebony	Manil/Matakki	Meranti, light red	Maple	Lime
Iroko	Massaranduba	Movingui	Oak, American red	Poplar
Jarrah	Meranti, dark red	Niangon	Okoumé	Ramin
Kapur	Mora	Sapeli	Seraya, white Sweetgum	Willow
Lignum vitae	Oak, European		Tulip tree	
Merbau	Oak, American white			
Muninga	Robinia			
Padouk	Sipo			
Peroba de Campos	Tola branca			
Purpleheart/amarante				
Teak				
Wengé/Panga panga				

Table 3 Natural durability of wood species

vacuum-pressure cycles is applied, forcing the chemicals into the timber. A vacuum is applied to remove air from the timber, then the wood is submerged in the liquid and a pressure is applied of up to $1.5\,N/mm^2$ to press the preservative into the timber, and finally another vacuum cycle is applied to remove excessive preservative that was not bonded with the timber. After the process, some time is needed for the chemicals to fix to the timber (see **Figure 7**) and to allow non-fixed chemicals to diffuse out of the timber for reuse and for environmental protection. Depending on the end-use of the timber and the perceived hazard level, a certain retention level is prescribed in terms of kg/m^3. The amount of preservative required to be retained to resist a given hazard level is normally specified in material standards or codes of practice.

Species name (botanical name)	Marine borer resistance
Basralocus (Dicorynia guianensis)	D
Cumaru (Dypteryx spp.)	D
Demerara greenheart (Ocotea rodiei)	D
Jarrah (Eucalyptus marginata)	D
Azobé	M
Teak (asian)	M

Table 4 Natural durability against marine borers

The higher the retention level, the larger the amount of preservative needed per m^3 of timber, the more severe the conditions to which the timber can be exposed. For many applications it is important that the treatment takes place after the members are cut to length and the holes for fasteners and connections are drilled. In this way the areas around the fasteners are treated and the timber member has what is referred to as a full envelope treatment. The reason for envelope treatments is that untreated parts of the members may become exposed when the structure is in place, creating a micro environment where rapid degradation can result. This is especially true for large glued laminated members that are difficult to replace when the structure is in service. An example of large glued laminated timber members subjected to a creosote envelope treatment is shown in **Figure 8**, taken during the construction period of the Flisa Road Bridge in Norway.

Application classes

Besides the natural durability, it is important for the material selection process to reflect the climatic conditions

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Figure 6 Installation for preservative treatment of wood

Figure 7 Stacks of untreated (left) and treated (right) wood

Figure 8 Creosoted glued laminated member in a truss joint of the Flisa Bridge, Norway

in which the structure will be placed. The climatic conditions not only influence the choice of material (wood species, treatment, level of prefabrication, etc.) but must also be considered in the design calculations, as the climatic conditions influence the material properties in terms of strength and stiffness.

For application in ground contact or water contact, timber with a natural durability of Class 1 or 2 has to be selected. Alternatively, a preservative treatment may lead to a similar resistance.

In **Table 5** an overview is given of application classes, ranging from dry indoor to wet outdoor in a marine environment.

As a help for designers using EN 460, a cross-reference table (**Table 6**) has been developed where the hazard class is related to guidance about the required species in terms of durability and/or preservative treatment.

Strength grades and classes

In order to build safe structures, the timber needs to be graded into quality grades and/or strength classes. As

Hazard class	Characterisation	Wetting	Approximate wood moisture content	Type of structure
Hazard class 1	Timber is in an indoor (heated) climate and fully protected from direct weather and wetting	Permanently dry	Yearly average around 12%, maximum 20%	Indoor climate, floors, purlins
Hazard class 2	Timber is under a roof, not exposed to direct wetting	Occasionally wet	≈ yearly average around 16%, maximum over 20%	Roof elements, unheated covered elements
Hazard class 3	Not covered and not in ground contact	Regular exposure to moist	≈ 16–20%	Façades, uncovered roofs
Hazard class 4	Ground contact, freshwater	Permanent exposure to ground or freshwater	Over 20%	Poles and piles, sheet pile walls, playgrounds
Hazard class 5	Salt water	Permanent exposure to salt water	Saturated	Water works in harbours, coastal areas

Table 5 Relationship between exposure threads and structure types

noted in previous chapters, the grading operation is normally done either visually or by machines, noting that machines generally work more accurately and faster and give higher yields in high strength classes. On the other hand, visual grading can be done anywhere and anytime, but the correlation between visual characteristics (knots, slope of grain, growth ring width, cracks, etc.) and strength is generally lower than for machine grading where density and stiffness are among the most common measured parameters in the grading process.

Timber assigned to a strength class or grade on the basis of visual or machine grading is marked so it can be traced and controlled during all times between sawmill and end-use, as noted in the example in **Figure 9**. In Europe the timber can be marked with the CE-mark. Generally, a distinction is made between softwoods and hardwoods. In Europe, C (coniferous and poplar) and D (deciduous) classes are given in EN 338 'Structural timber – Strength Classes'. C-classes range from C14 to C50 and D classes range from D30 to D70. The number indicates the characteristic value of the bending strength. Once assigned to a strength class, all strength and stiffness values for tension, compression, shear and bending are fixed. A list of species and grades that are assigned to strength classes can be found in European Standard EN 1912 'Structural Timber – Strength Classes – Assignment of Visual Grades and Species'.

Australian grades are given as S1 to S7 for green timber and SD1 to SD8 for dry timber. Canada and the US and many other countries have their own systems for specification of grades, for instance Select Structural, No. 1, No. 2 and No. 3. Generally, for standard wood frame construction the grades for the different components (studs, planks, floor boards etc.) are prescribed in specific guidelines (see http://www.cwc.ca, www.spib.org).

It is important for structural engineers to note that the values attached to the strength classes in use in a certain country are always related to the design method. This may be a design method based on allowable stresses or on a load and resistance factor design (LRFD) method. However, even if values are published as 'characteristic values', they cannot be exchanged between LRFD design

Hazard classes	Natural durability class				
	1	2	3	4	5
1	O	O	O	O	O
2	O	O	O	(O)	(O)
3	O	O	(O)	(O)–(X)	(O)–(X)
4	O	(O)	(X)	X	X
5	O	X	(X)	X	X
O	Natural durability is satisfactory				
(O)	Natural durability is in principle satisfactory, but some circumstances may require timber to be treated				
(O)–(X)	Natural durability could be sufficient, but timber species, treatability and the final application may lead to chemical treatment				
(X)	Chemical treatment is advisable, but for some applications the natural durability may be satisfactory				
X	Chemical treatment unavoidable				

Table 6 Guidance for choice of wood species and species treatment

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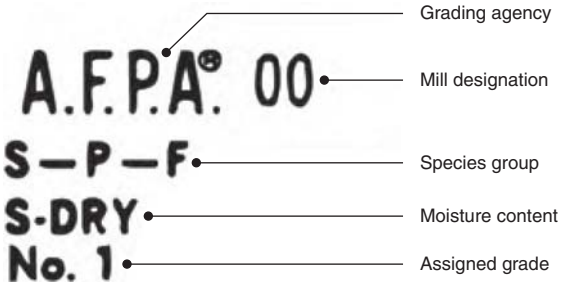

Figure 9 Example of marking (courtesy of the Canadian Wood Council)

systems without a check on the method used to determine the characteristic values and how these are applied in the design procedure specified in the relevant design codes.

Surface finishes and sizes

In addition to taking account of specifications for species, durability, grades and strength classes, designers are often required to specify the required surface finishes and sizes of the timber. Surface finish relates to a planed or non-planed surface of the timber but may also deal with the wood colour and wood quality on the visible side of the members. Glued laminated members and cross-laminated timber panels may have special surface boards for visual appearance.

Both sawn timber and standard glued laminated timber are produced in standard sizes in most markets. For sawn timber, distinction is made between rough sawn sizes and planed sizes, the latter generally taking 4 mm off from the rough sawn width and height. Deviations from target sizes are allowed to some extent and also allowable deformations with regard to bow (a), twist (b), spring (c) and cup (d) as shown in **Figure 10**. Often, the allowed deformations are related to a measuring length of 2000 mm for bow, twist and spring and to 25 mm for cup. Tolerances for acceptable variations in geometric dimensions and finish are normally specified in the relevant product standards.

Functionality is a special subject that must be considered during the design process. Hand-railings for bridges are a good example. Splintering is not allowed in such cases and timber with long parallel grains is preferred. Examples of such species are Bangkirai (*shorea* spp.), Basralocus (*Dicorynia* spp.), Demerara greenheart (*Ocotea rodiaei*), Ipé (*Tabebuia* spp.) but softwoods such as Douglas fir (*Pseudotsuga menziessi*) with appropriate treatment may also be suitable.

Service life performance and design for durability

It is essential for designers to have a sound grasp of both the expected hazards and required material specifications to

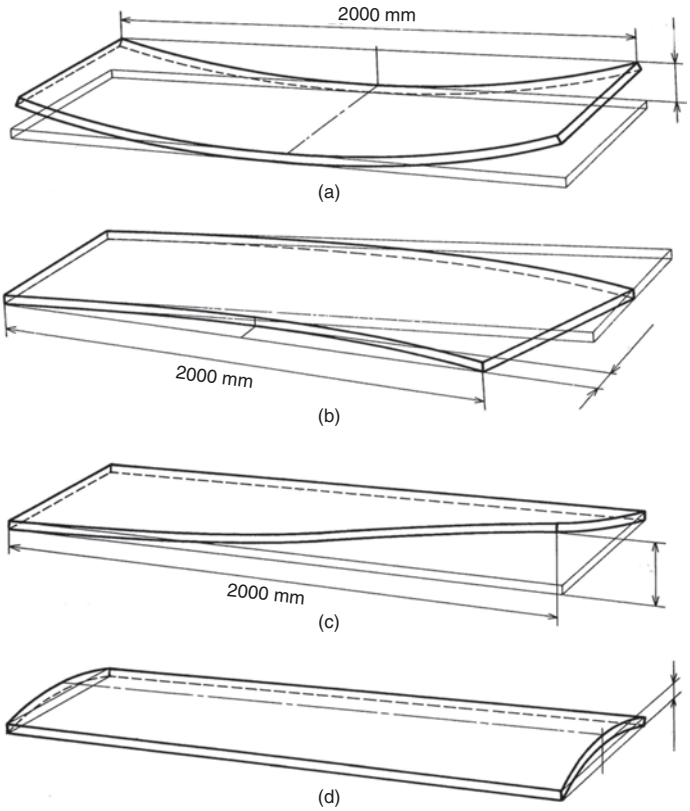

Figure 10 Deformations in timber boards

ensure that a structure will remain functional where a long service life is expected. However, these two criteria are not enough by themselves. The most important service life performance factor is good structural detailing, followed by the correct choice of wood species and/or preservative treatment.

The choice for a durable hardwood or treated softwood is an important consideration in obtaining a long service life. However, such a choice is no guarantee of appropriate long-term performance. Most indoor applications of timber tend to be able to achieve the required service life with very little maintenance. Outstanding examples of structures that have been successful in achieving a long-term service life are covered wooden bridges, which can be found across Europe and in North America (McKee, 1997). One such case is the covered bridge in Bassano del Grappa in the north of Italy, designed in 1569 by Architect Andrea Palladio, as shown in **Figure 11**. Because of the roof cover, critical structural timber members and decking are not exposed to direct rain and sun, increasing the service life performance. Other examples are buildings with large eaves that avoid direct weathering to a large extent. In other words: keep and let the timber remain dry is the main message. A good design with a long guaranteed service life requires:

Figure 11 Covered timber bridge in Bassano del Grappa, Italy from the year 1569

Figure 12 Covered cantilevered glued laminated timber beams to avoid direct rain and sun exposure, Mezzolombardo, Italy

- a good assessment of the exposures and hazards (biological, mechanical, physical and chemical);
- the right choice of wood species and/or timber treatment;
- careful choice of fasteners and fastening techniques;
- a good structural design with appropriate structural details;
- detailing that allows for the timber to dry out after wetting.

In addition to these five critical points, a sixth may be added which is related to the production and handling process of the timber. It is particularly important that the wood is dried under controlled conditions to the right moisture content, i.e. close to or at the moisture content it will acquire in the final application, and that it is stored and transported correctly and the surface finish is in accordance with to what has been agreed upon.

At the current time Australia is the only country that has developed a comprehensive system for service life prediction of timber in structures, where calculation methods are available for predicting service life performance for given structural applications, taking into account the reduced reliability during service life by degradation of the members (Leicester, 2001; FWPA, 2007). In most other countries, more prescriptive codes and textbooks are used in an attempt to ensure a durable timber application with an acceptable service life. Examples of good structural details for a wide variety of structural applications can be found in Herzog *et al.* (2004). Details for façades can be found in design handbooks such as Holzfassaden (2004).

Examples

The best way to illustrate good practice is to refer to good examples. **Figure 12** shows a glued laminated structure with cantilever beams. The beams are covered by zinc-coated plates and the roof structure is made of transparent Plexiglas panels. The cantilever beams are tapered and not exposed to direct rain and sun. In the same structure, a header beam has been placed slightly rotated about the longitudinal axis, also to avoid direct exposure, as shown in **Figure 13**.

A portal frame with moment-resisting joints is shown in **Figure 14**. The portal frame is for the major part covered by the roof structure. Additionally, the frame structure is treated with a surface paint. Columns that are exposed to rain and direct sun should be elevated from the ground floor by at least 300 mm, as seen in **Figure 15**. In this way,

Figure 13 Header beam, rotated about the longitudinal axis to avoid direct exposure, Mezzolombardo, Italy

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Figure 14 Portal frame with surface treatment covered by a roof structure, Lahti, Finland

Figure 16 Spacer between railing and main girder to allow water and dirt to run-off easily, Haasnoot Bruggen, the Netherlands

Figure 15 Durable design for a column base

rainwater can easily run off and splash water is not able to reach the column.

A good detail of a bridge railing is shown in **Figure 16**, where the bridge railing is fixed to the steel girder with 40 mm spacers, to avoid dirt accumulation and allow water to run off easily.

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Chapter 65

Case studies: current uses of timber in the UK

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Wood is one of the most versatile building materials. It can be used effectively in all construction applications, from façade, windows and internal lining to load-bearing structures, such as framing, roofing or large-span spanning members. This chapter demonstrates the potential for timber construction forms, aesthetically, economically, environmentally and in its structural load-bearing ability.

Introduction

This chapter demonstrates a wide range of applications of timber in UK construction. It is not intended to be a comprehensive catalogue of timber uses to date, but rather to show a selection of typical types of buildings and design scenarios where timber has been used successfully. The projects chosen showcase the potential of timber aesthetically, economically, environmentally and in its structural load-bearing ability. Its aim is to provide the construction professional with an initial introduction to timber construction and some of the practicalities to consider. The following three examples of timber applications are covered:

- one-storey, single-family dwelling;
- multi-storey, multi-occupancy apartment building;
- large-span visitor centre.

Each case study discusses the background to the project, why timber was used and what the specific drivers were for using this material. Each case study lists some of the timber and other structural materials used in the structure, giving useful reference to currently available products and providers of specialist support. The structural concept and special considerations for performance areas such as sustainability are outlined.

The case studies demonstrate the practical application of timber but of course they are not exhaustive, so additional information and background can be found in the following box:

General information on wood and wood products

- Tackle climate change: Use Wood, CEI Bois, 2006 – also available for download at http://www.cei-bois.org
- Desch H. E. and Dinwoodie J. M. *Timber-structure, Properties Conversion and Use*, 1956, 7th edition.

Further reading, codes for timber design and execution

- BS EN 1995-1-1: 2004. Eurocode 5: Design of Timber Structures Part 1-1: General – Common Rules and Rules for Buildings.
- NA TO BS EN 1995-1-1: 2006. UK National Annex to Eurocode 5: Design of Timber Structures – Part 1-1: General – Common Rules and Rules for Buildings.
- BS EN 1995-1-2: 2004. Eurocode 5: Design of Timber Structures Part 1-2: General Structural Fire Design.
- NA TO BS EN 1995-1-2: 2006. UK National Annex to Eurocode 5: Design of Timber Structures – Part 1-2: General – Structural Fire Design.
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- Grantham R., Enjily V. *et al*. *Multi-storey Timber Frame Buildings: A Design Guide*, BR454, BRE, UK, 2003 Timber frame construction, 4th edition, TRADA Technology Ltd, 2008.
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Case study 1: the 'Lighthouse'

Kingspan set out to build the first zero carbon house in the UK. Zero carbon, as defined by Level 6 in the *Code for Sustainable Homes* (Communities and Local Government, 2008 – www.communities.gov.uk) is the standard specified by the UK government to which all new homes are to be designed and constructed by 2016. The experimental Lighthouse one-family dwelling (see **Figure 1**) has strived to achieve this and the other associated requirements of Level 6 by addressing aspects of building design covering the use of sustainable materials, energy consumption and CO_2 emissions, water use, waste, pollution and surface water run off, health and well-being, management and ecology.

The Code, with no specific level requirement, was made mandatory in England from May 2008. It aims to ensure that homebuyers are provided with information on the sustainability performance of their properties. Houses built to the Code are designed to have lower running costs, improved well-being and reduced environmental impact. The Lighthouse design team has trialled design

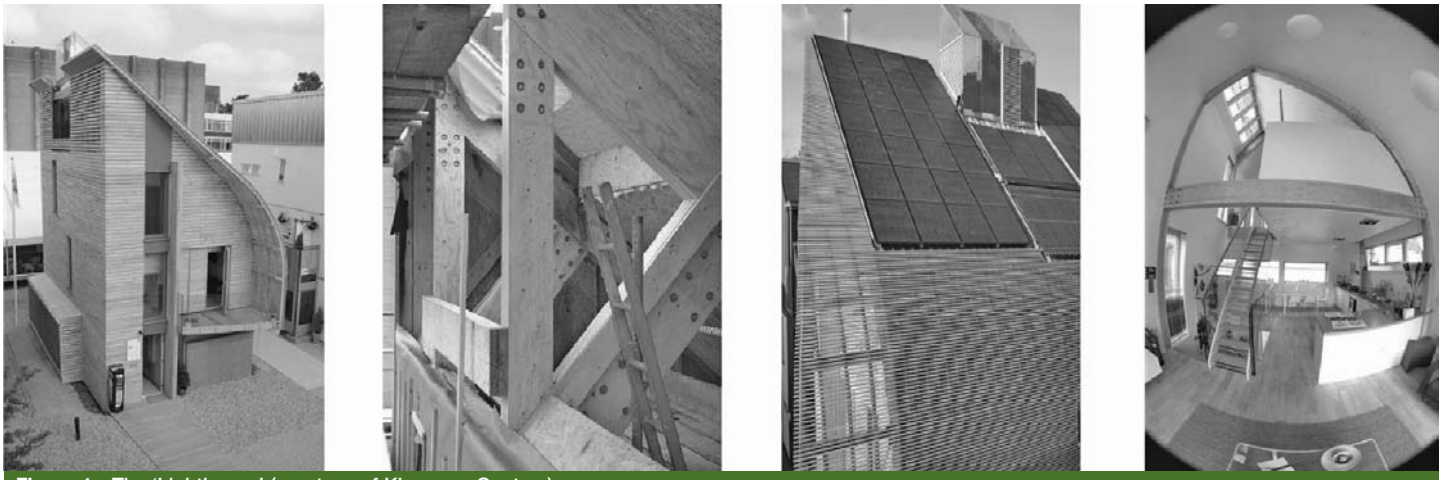

Figure 1 The 'Lighthouse' (courtesy of Kingspan Century)

methods and technologies that enable zero carbon[1] energy performance of the occupied home. To achieve this, the impact and advantages of the use of timber components and products have been considered in detail. This was of particular importance to achieving the heat loss value prescribed for 'zero carbon homes' of 0.8 W/m^2K or less. Reaching this performance level is governed by the thermal performance of the building fabric (walls, roof, windows) and also requires adequate performance levels for airtightness and air exchange.

Why timber?

For the Lighthouse to become an exemplar for sustainable construction to the Code for Sustainable Homes (see **Figure 2**), a set of sustainable design principles covering performance in nine key areas have to be considered:

Materials	Surface water run-off	Heath and well-being
Energy and CO_2	Waste	Management
Water	Pollution	Ecology

The performance of a building is assessed in the nine categories shown in the table. Some of the credits relate to the setting of a development/building, whereas others consider the building fabric and materials used in a building. There is a mandatory requirement to achieve a performance rating for at least three of the following five elements of the building envelope:

- roof;
- external walls;
- internal walls (including separating walls);
- upper and ground floors (including separating floors);
- windows.

Between 1 and 15 credits are available depending on the sustainability ratings (see Anderson and Howard, 2009) and relative distributions of different materials across the five main elements of the building envelope. Using carbon-storing timber and timber-based products allows you to collect high ratings and therefore high credits in the material category listed above. A further advantage of timber products is the availability of prefabricated, quality

Figure 2 The Code for Sustainable Homes, Communities and Local Government, December 2006

[1] In a 'zero-carbon home' the net carbon dioxide emission resulting from all energy used in the dwelling are zero or better. This includes the energy consumed in the operation of the space heating/cooling, hot water systems, ventilation, all internal lighting, cooking and all electrical appliances.

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Figure 3 Examples of prefabrication in domestic housing: (a) pre-manufacture: quality control, energy and waste minimization; (b) trends in domestic construction using timber materials and components (courtesy of Kingspan Century)

controlled, highly insulating building solutions (see **Figure 3**) ranging from external to internal walls, floors and roofs. This enables precise, airtight construction on site. The offsite production of components also helps to control and monitor energy use and waste generation during production.

Structure and concept

The structure of the 93 m^2, two-and-a-half storey, two-bedroom house is a simple barn-like form, derived from a 40° roof, which accommodates an array of photovoltaic panels. The sweeping timber roof envelops the central space – a generous, open-plan, top-lit, double-height living space, with the sleeping accommodation at ground level. The living space uses a timber portal structure so that floors can be slotted between the frames or left open as required. This highly insulated, airtight building fabric was designed to provide generous daylight levels and includes effective solar control.

Stability is achieved through traditional timber moment connections at first floor and ceiling level. At ground level a timber frame structural layout carries the vertical loads of the open-plan frames above and provides stability to the load-bearing shear walls. These are constructed using Kingspan Off-Site's TEK Building System, a high-performance timber-based wall system. It provides a high level of thermal insulation and performance. With U values of 0.11 W/m^2K and airtightness rates of less than 1.0 $m^3/h/m^2$ at 50 Pa, heat loss is reduced by potentially two-thirds of that of a standard house. The foundations consist of offsite timber floor cassettes on a ring beam of timber beams supported off the ground level by lightweight screw-pile head foundations. The piles provide minimal disturbance to the ground and provide suitable supports for domestic-scale dwellings. When the building reaches the end of its useful lifespan, the foundation support points can be removed.

Products and materials

A range of products and materials is used in the Lighthouse. The structural envelope is constructed with a structural insulated panels (SIPs) system, a timber frame derivative. SIPs are prefabricated lightweight building units used as principal load-bearing components in domestic and light industrial construction of up to three storeys in height. This construction method, which originated in the US, is now increasingly common in the UK building industry. In Europe, SIPs are mainly used as load-bearing internal and external walls and sometimes roofs, whereas in the USA they are also frequently employed as floors and roofs. A SIP consists of two high-density face layers which are bonded on both sides of a low-density, cellular core substrate. A strong, structural bond between the three layers is essential to the load-bearing ability of the SIP so that high loads can be transmitted by the relatively light units without using internal studding (see **Figure 4**).

The portal beams used to create the open space on the first floor are LVL (laminated veneer lumber). LVL is an engineering wood product, in which timber layers are glued in orientation of the fibre to create defect free, dimensionally stable, load-bearing units. This material is typically used in structural applications, trusses, beams and rafters. It can also be used as a bracing element. LVL is manufactured in a continuous process, using a resin to bond the timber layers. The material is suited for pressure treatments and can be designed for applications in high-risk biological attack (e.g. by fungi or insects) areas or where adverse climatic conditions prevail.

The first floor is constructed from timber web joists using cement-bonded particle wood-based boards. A floating timber cassette ground floor, replaces a concrete slab. The cassettes are constructed from timber I-joists and OSB (oriented strand board) with full foam insulation in the

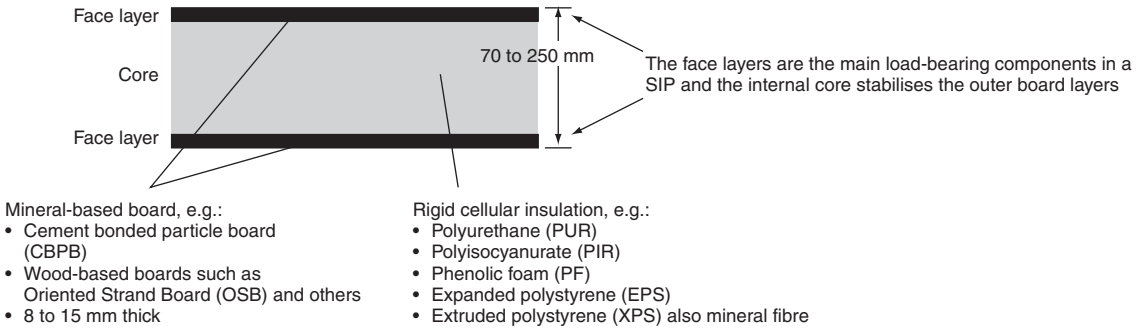

Figure 4 Cross-section of a typical SIP

voids. The cassettes are joined into a timber rim beam which connects to the lightweight screwpile foundations. The structural design used the UK's current timber design code BS 5268, Parts 6.1 and 6.2 and parts of EN 1995-1-1.

The Lighthouse also tested examples of 'thermal heavyweight' room ceiling surfaces within a lightweight structural system. Two different surfaces were used: a phase change materials plasterboard (Tradename: BASF PCM (Phase Change Material) and dense cement fibre board. These surfaces are to help absorb daytime heat and to release stored heat during cooler night periods to purge ventilation. Phase change materials absorb room heat by changing from solid to liquid within microscopic capsules embedded within the board. This process is then reversed when the room is cooled with night air. The effectiveness of these materials is trialled in the Lighthouse – an assessment of their effectiveness is currently ongoing.

The Lighthouse is clad in untreated sweet chestnut, the heartwood of which is Durability Class 3 (see also BRE, 1998), with an expected 30-year design life. Timber cladding is used increasingly in a variety of patterns, textures and colours, in conventional boarding but also with open joints, as in the Lighthouse. The choice of species is influenced by the expected design life and increasingly by their environmental and certification credentials. The choice of the cladding solution and the approach to ensuring adequate durability, using timber of sufficient natural durability or protection through the use of preservatives, wood modification technologies (BRE, 2007) and/or coatings are important to consider at the design stage. Surface finishes and other timber modification also need to be reviewed in terms of fire performance. Fixing methods and detailing should accommodate moisture movement resulting from seasonal changes and not provide opportunity for corrosive action.

Additional information

The property includes solar control and sustainable technologies, water efficiency techniques, renewable energy, passive cooling and mechanical ventilation and heat recovery (MVHR). This provides the user of the home with an adaptable, flexible space designed for modern living, intuitively integrating sustainability. Further incorporated concepts include reduced glazing – ratio of glazing to wall is 18% as opposed to 25–30% in a conventional house – and high level of thermal efficiency and airtightness. The reduction of use of water is also central to attaining high code level ratings and in the Lighthouse design, around 30% of the water is provided from rainwater harvesting or water recycling systems. The house incorporates smart metering and monitoring systems to aid occupants so they do not consume more than around 80 litres of potable (drinkable) water per person, per day. It is estimated that the Lighthouse has 50% reduced water costs compared to a conventional house. Wind catchers provide secure night-time ventilation, and energy costs are closely monitored and estimated at £31 per year (in 2007). The innovative foundation system enables minimal ground disturbance and in 2007 the house would currently be exempt from stamp duty.

Client: BRE Innovation Park
Architect: Sheppard Robson
Structural Engineers: CCB Evolution, Arup
Collaborators: Kingspan, Davis Langdon, MacFarlane Wilder, Chorus

Case study 2: multi-storey timber structures

The timber frame market in the UK is vibrant and growing. With shares of up to 57% (National House Building Council, 2006) in parts of the UK, the timber industry plays a significant role in the new build sector and has become a strong competitor to the more traditional forms of construction.

Why timber?

Timber framing is enjoying an increased popularity because of the efficiency of offsite fabrication and the potential to reduce CO_2 impact with timber-based solutions. This has led to rising interest in the use of timber as a key element

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Beck Street, Nottingham, UK

Trinity Village, Preston, UK

Ion building, London, UK

Figure 5 Examples of multi-storey timber buildings

in construction and the applications now range from the main structural frame to the external façade finishes, in the more traditional two-storey domestic market but also more widely in multi-storey applications, for both domestic and non-domestic applications. Light frame construction for homes, hotels and multi-storey accommodation buildings is an increasingly common form of construction in the UK.

In the past 15 years the timber frame industry has expanded market share in private and social housing, multi-occupancy domestic (flats) and commercial projects for hotels, nursing homes, student accommodation and other commercial buildings. Four-storey timber frame buildings have been constructed extensively since the mid-1980s. The use of pre-assembled wall and floor panels to create a fast and accurate structural building envelope has evolved from the two-storey house to five, six and seven multi-storey timber framed (MSTF) buildings (see **Figure 5**).

Traditionally timber frame construction uses small-section timber components, with typically minimum dimensions of 38 mm × 72 mm, in combination with panel products, such as OSB, plywood and plasterboard. The UK supply chain for low-rise timber panel construction is well established and has had significant success since the 1960s. This existing supply chain has been attracted by the growing market for five-, six- and seven-storey multi-storey buildings, as the use of timber frame in these applications offers advantages as manufacturing efficiency increases (see **Figure 6**).

Strategic research has been a catalyst for the growth of multi-storey timber frame in the UK. An extensive investigative programme on an experimental six-storey timber frame building was carried out by BRE (Building Research Establishment) and TRADA (Timber Research Establishment) in the late 1990s. The research work included full-scale model testing at BRE's Cardington facility, generally referred to as 'TF2000', covering work on structural and fire performance as well as erection processes and productivity benefits.

Structure and concept

The use of small section timbers and panel products such as OSB, plywood and plasterboard lining to form pre-assembled structural walls, floors and roofs involves many structural interfaces that are well documented and understood. The issues around this are not influenced by storey heights except in that the loadings and number of studs may increase. Below are some of the special structural issues to be considered when using timber and timber-based products in multi-storey applications.

Vertical load-bearing capacity

The vertical load-bearing capacity of a timber stud is dependent on its cross-section, the limiting capacity of compression perpendicular to the grain of the horizontal supporting members and on the lateral restraint available within the assembly. In higher-rise applications the increased vertical loading on a stud wall generally results in an increase of the volume of timber in the wall. The volume of studs in a wall panel can become problematic from a manufacturing and also a thermal performance point of view.

While not a conventional structural design consideration, the calculation for frame shortening in a timber frame is essential for engineers using timber. With increasing floor levels there is an increase in the accumulative cross-grain movement in the frame and panel-to-panel junction interfaces. Significant shortening can occur due to shrinkage of timber. In the UK this movement is often referred to as differential movement. It is the movement variation of the loaded timber frame structure to that of elements that are not vertically supported by the frame, such as masonry cladding. The closing up of joints under load also produces a small amount of movement as does the often negligible contribution of elastic deformation and creep. It is important for structural engineers to consider these issues when using timber products and also to understand the impact

Figure 6 Example of a multi-storey timber frame development (courtesy of CCB Evolution)

of changing moisture contents, load durations and the tolerance of the materials used.

Resistance to horizontal forces

Horizontal resistance of a timber frame panel in its plane, also termed 'racking resistance', is used to resist shear forces and is an important design parameter in timber frame construction. The design procedures are documented in BS 5268 Part 6.1, a document widely used by the timber engineering community. With the introduction of EN 1995-1-1 in 2010, the empirical values given in the UK code will be superseded and this will cause a shift in design principles; the exact design procedures are currently still under discussion. Timber-based structural solutions in the UK are generally sensitive to racking design procedures, highlighting the importance of the use of inherent in-plane shear resistance of the structural external walls in multi-storey construction.

Robustness and disproportionate collapse

Structural robustness becomes an increasingly important design element in the design of taller buildings. While all buildings are expected to perform safely, for standard loading scenarios additional performance requirements are to be met for taller buildings. Tall buildings have to exhibit inherent additional strength to cope with accidental loading situations. Multi-storey timber construction is required to be designed against progressive collapse. Design against disproportionate collapse is to ensure that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause of the accident.

The UK has adopted the principles of EN 1991-1-7: 2004 Annex A, and in particular the use of Table A.1, which is reproduced as Table 11 (see **Table 1**) in the UK Building Regulations (Approved Document A, A3). The Regulation provides a class system to determine the risk category for a building. For typical timber frame buildings, categories 1, 2A and 2B are relevant, with category 2A for buildings up to four storeys and category 2B for those in the five-, six- and seven-storey range. Different design considerations need to be considered when a building moves from category 1 to 2A, and from 2A to 2B. BS 5268 Part 2 provides engineering guidance and rules on how to comply with each design category in structural design.

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Class	Building type and occupancy
1	Houses not exceeding 4 storeys
	Agricultural buildings
	Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1.5 times the building height
2A	Five-storey single-occupancy houses
	Hotels not exceeding 4 storeys
	Flats, apartments and other residential buildings not exceeding 4 storeys
	Offices not exceeding 4 storeys
	Industrial buildings not exceeding 3 storeys
	Retailing premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey
	Single-storey educational buildings
	All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey
2B	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys
	Educational buildings greater than 1 storey but not exceeding 15 storeys
	Retailing premises greater than 3 storeys but not exceeding 15 storeys
	Hospitals not exceeding 3 storeys
	Offices greater than 4 storeys but not exceeding 15 storeys
	All buildings to which members of the public are admitted which contain floor areas exceeding 2000 m^2 but less than 5000 m^2 at each storey
	Car parking not exceeding 6 storeys

Table 1 Extract from the UK Building Regulations 2004: Approved Document A (A3), available at: http:// www.planningportal.gov.uk

Products used

A multi-storey timber frame structure generally employs timber frame walls for external, internal load-bearing and non-load-bearing applications. The timber grade is generally C24 and for external walls the timber is preservative treated for durability. Internal walls are generally untreated. Sheathing materials most commonly used are OSB (grade 3 or 4) and plasterboard of different thicknesses and additional sacrificial lining material to meet fire resistance requirements. Sheathing grade plywood, medium board and impregnated soft board and other materials such as tempered hardboard, moisture-resistant chipboard and cement-bonded particle board are also used.

Ground floors are generally in concrete while upper floors are increasingly completed in engineered wood floors (also referred to as 'I-beams'), which are assembled and connected to the next level of construction with joist hangers. Chipboard, plywood, OSB, cement-bonded particle board and tongued-and-grooved softwood boards are commonly used timber-based floor decking materials. The most commonly used ceiling lining is gypsum plasterboard. Stairshafts and stairs can also be completed in timber. Special fire resistance requirements apply in these cases and a fire performance assessment needs to be undertaken.

Design codes

Dedicated guidance for multi-storey timber construction has been minimal. A joint BRE and TRADA publication entitled *Multi Storey Timber Frame Buildings – a Design Guide* published in 2003 is the only document that provides reference to buildings above four storeys (BRE/TRADA, 2003). Until November 2007 the British Standard BS 5268 Part 6.1 covered specific aspects of timber frame limited to four-storey height. The new revision now addresses buildings up to seven storeys, incorporating a number of key changes that address the specific structural issues relating to multi-storey timber frame buildings. It also formalises best practice and design guidelines to ensure timber frame construction is not applied to unrealistic layouts and schemes.

From 2010 the timber design community will be converting to EN 1995 *Design of Timber Structures*. It has three parts:

- EN 1995-1-1, General – Common Rules for Buildings;
- EN 1995-1-2 General Rules – Structural Fire Design;
- EN 1995-2 Bridges.

The introduction of EN 1995 will have a marked impact on the UK timber design community, the main difference being the conversion from permissible stress to limit state design. UK-specific non-contradictory complimentary information will remain. The scope and extent of the guidance is currently being developed.

Case study 3: Savill Building, Windsor Great Park

Situated in Windsor Great Park, the roof of the Savill Building is the UK's largest timber gridshell structure (**Figure 7**). Buro Happold provided the structural engineering design for the roof of the building, which is a spectacular, graceful, three-domed undulating timber structure, 90 m long by 25 m wide. The brief from the client, The Crown Estate, was to build a structure that would be dramatic yet sensitive to its surroundings. Buro Happold engineers worked closely with Glenn Howells Architects and woodwork specialist, the Green Oak Carpentry Company (GOCC), to successfully create the structure from timber that was sustainably harvested from Windsor Great Park's own forest.

Based on the naturally tough structure of a shell, a gridshell construction was chosen for its strength and flexibility. Created to mirror the skyline of the surrounding trees, the exterior of the roof follows a flowing curve. Looking at the timber gridshell's gentle curve from the inside of the structure, the visitor has an alternative but equally impressive view of the building which gives them a remarkable insight into the way the roof works, proving that a

Figure 7 The Savill Building, Windsor Great Park (courtesy of Burro Happold)

building's structure and interior can be as graceful as its exterior.

Why timber?

Completed gridshells are strong, but while under construction the individual elements need to remain pliable. This allows for the structure to be formed into a shape and then locked in the final desired form. The components of the shell are laid out on a flat platform and then carefully raised or lowered into the required shape. For the Savill Building roof this was achieved through a bespoke scaffolding system through which the height of the roof was adjusted at 200 points across the plan area.

Timber is the ideal material for this type of structure as it is lightweight, flexible in shape and connection design. Its adaptability with other structural materials enabled the building of the dramatic structure as envisaged by the architect. It includes high openings into the surrounding gardens. The oak used was specified for its natural resistance and for the silvery-grey weathered appearance. More than 20 km of 80 mm by 50 mm 'finger-jointed' larch timber has been used in the gridshell which only weighs 30 t.

Structure

The roof is a three-domed, double-curved structure of sinusoidal shape, visible on the inside of the building. This is the latest in a series of timber gridshells that Buro Happold has designed, continuing a lineage of lightweight, environmentally sensitive structures.

The geometric form of the Savill gridshell relates back to a single setting-out line, which is very near to the building's centreline and is an arc in plan. Along this line, the roof cross-section is set out as a series of parabolic curves. In the vertical plane, the centreline rises and falls on a sinusoidal curve. Load concentrations on the structure were a key consideration. Proposals for an all-timber structure were reviewed but, in being true to the design competition concept and creating a dramatic structural statement, the long, high openings into the garden led to the introduction of steel tubes for the perimeter ring and the quadruped legs, as illustrated in **Figure 8**.

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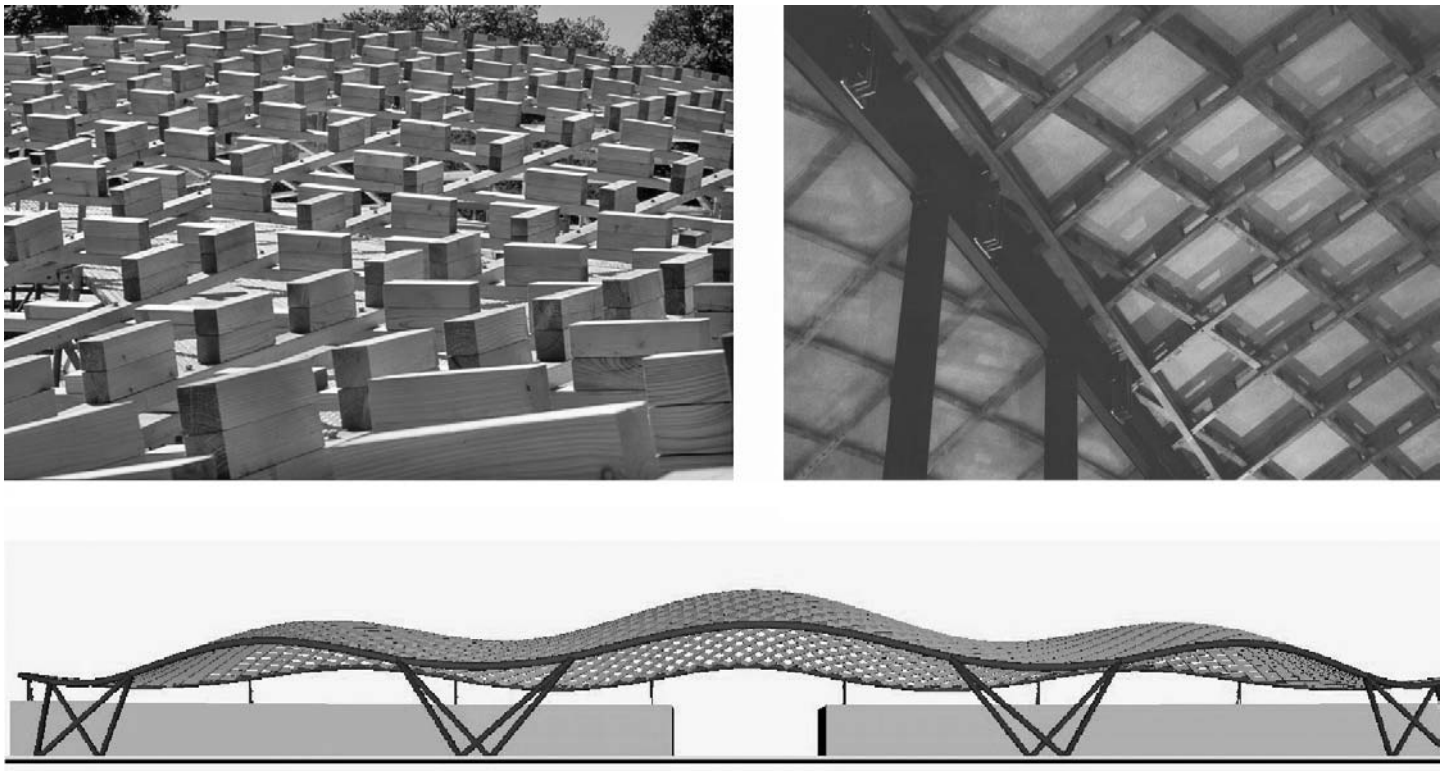

Figure 8 Structural form of the dome roof for the Savill Building (courtesy of Burro Happold)

The timber structure rises from a perimeter ring. Kerto LVL fingers, bolted to the steel perimeter, are used to transfer load from the larch laths shell to the legs. The LVL fingers are hidden behind a soffit of plywood extending beyond the glazed perimeter walls but, in places of very high load concentration, they can be seen pointing into the interior gridshell space. To act as a shell, the structure must be strong and stiff in its plane. Initially the concept included steel cables to triangulate and thus brace the shell in its plane. To save cost and make a more elegant structure, the cables were omitted and plywood covering was used instead. The Savill gridshell is made up of a regular 1 m grid of 80 mm × 50 mm sections of larch timber. The three-domed shape is clad in oak.

The structure's own weight is easily carried by the timber and, with no other loads applied, the stresses in the laths and the plywood bracing are small. More critical are the forces induced by severe wind and snow. In these design situations, the structural plywood helps transfer the forces through the peaks and valleys of the roof, to the steelwork and foundations.

Products used in the structure

A number of timber species were considered for use in the gridshell, including larch, Douglas fir, chestnut and oak. They were selected for their inherent natural durability. Furthermore, these timber species were readily available from sustainable sources from the Crown Estate. All the timber used in the structure has come from the Crown Estate's commercially managed (and Forest Stewardship Council certified) woodland in Windsor Great Park.

This sourcing process, which began in 2003, was carried out in parallel with a series of structural tests that were conducted on the wood to characterise its structural properties. The tests informed the structural design process and were critical in determining lath size and spacing as well as the quality of the bolted joints and screwed shear blocks. Further selection of the wood into high- and low-grade timbers was made to ensure that critical structural members, such as the long lengths which carry the internal loads to the perimeter, are of necessary strength. Other elements, such as the infill blocks, are made of lower-grade timber.

Previous experience (the Downland gridshell) indicated that the performance of the oak exceeded that of the other species. While it was stiffer than the other timbers tested, needing a larger force to achieve a given curvature, it had a higher bending strength, achieving a smaller bending radius prior to failure. In addition the failure mode was not sudden: there was a degree of plasticity.

Due to its acidity, oak is a difficult material to joint with adhesives, especially when green. The green oak used for the Downland gridshell had moisture contents of up to

65%. Swiss adhesive manufacturer Collano has developed a one-part polyurethane liquid adhesive under the trade name Purbond HB 530, which is not adversely affected by the acidity of green oak. This adhesive was designed to cure under the influence of material moisture and humidity.

The timber was optimised using the GreCon Dimter OPTICUT 101 mechanised saw. Each length of timber was inspected by a skilled carpenter to identify knots, unacceptable slope of grain and other defects. It was then graded using a fluorescent crayon which can be read by the saw's optimisation computer to cut out the defect and sorts, in accordance with the designated grade.

The Savill Building gridshell lattice required 20 000 linear metres of lath; as individual pieces of graded lath averaged 2 m in length, this represented 10 000 finger-joints. Finger-jointing was performed using the GreCon Dimter SUPRA finger-jointing machine. Fingers were cut simultaneously into the ends of the timber segments and the Collano Purbond HB 530 polyurethane adhesive was applied. The segments were aligned and jointed under pressure to form 6 m lengths.

Design code used

The laths and finger-joints were tested in a four-point bending test in accordance with sample dimensions given in BS EN 408 (British Standards Institution, 2003). The samples were tested green and not conditioned to the requirements given in BS EN 408: 2003. At the early stages of the project, a whole series of finger-joint testing was undertaken, examining a range of variables including capacity about both axes, performance of different adhesives and effect of different production pressure.

For the production stage, a quality control system was implemented to ensure the effectiveness of the finger-joints. Quality control testing was carried out in two stages, pre-production tests and batch sampling. The results of these tests enabled a statistical analysis in accordance with BS EN 384 (BSI, 2004). The pre-production tests determined the 5-percentile characteristic bending strength of the finger-joints, and coefficient of variation for the running production control. Testing during production ensured that the probability of accepting a sample with an ultimate bending strength was within acceptable statistical limits set out in BS EN 385. The structural design was completed following EN 1995-1-1 and EN 1995-1-2 and BS 5268.

Awards for the Savill building

This stunning, innovative structure was recognised for its outstanding design and craftsmanship in wood, when it swept the board at the 2006 Wood Awards. As well as winning the coveted Gold Award, the gridshell was also a winner in the Commercial and Public Access category as well as the Structural category. It was also shortlisted for the Stirling Prize and won the IStructE Supreme Award for Structural Engineering Excellence.

Client: The Crown Estate
Architect: Glenn Howells Architects
Services provided by Buro Happold: Structural Engineer

Summary

These case studies show the versatility of timber as a structural building material. They demonstrate the broad range of timber applications from sustainable, versatile load-bearing structures, to applications in higher-rise construction and aesthetical, award-winning long-span projects. The case studies also illustrate the potential of timber when used with other structural materials as well as the importance of detailing and design, which can encompass and unite performance and environmental considerations.

With the introduction of Eurocodes, where design procedures for all structural materials are now available utilising a uniform safety concept, there is great potential for more and more engineers to discover the possibilities of structural timber. It is hoped that construction professionals and their clients will increasingly exploit this sustainable and versatile resource by re-inventing traditional timber techniques in new contexts to create successful structural solutions.

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Section 9: Glass

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Chapter 66

Glass in construction: an introduction

John Colvin JCGC Limited, UK

Glass is different from any other material used in construction, as its mechanical behaviour is unlike any other. It is also transparent. At its best it can transform an ordinary building into a magic place in a way that no other construction material can. At its worst it is nothing but a nuisance, causing seemingly endless problems.

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Light relief

Glass is fascinating: glass is frustrating. Anyone who becomes involved with glass in a technical sense very quickly adopts one or other of these attitudes. You get hooked or you loathe it.

It is a material which obeys the rules, so it should be very simple to form and use, but because it obeys the rules it can be maddeningly difficult to form and use.

When liquid, glass is a Newtonian fluid – it flows in a manner that obeys all the simple rules for fluid dynamics developed many hundreds of years ago. That should make it easy to process since it follows all the simple rules for fluids. Yet a Newtonian fluid is one that flows without any turbulence. It is similar to how syrup behaves at room temperature. Now think about trying to get syrup to go where you want it to, but it is between 1100°C and 1700°C and you are struggling to find materials that won't burn, melt or disintegrate while you're trying to work it.

When solid, glass is a perfectly elastic, homogeneous, isotropic material. Delve deep enough into the engineering textbooks and you find that all the basic formulae for beams, plates and anything else you would want to design are developed from the presumption that the material used is (you've guessed it) perfectly elastic, homogeneous and isotropic. Glass obeys all the rules exactly. But instead of making life simple, it results in a material that is brittle, highly notch sensitive and very susceptible to localised overstressing – an engineer's nightmare.

Glass is light. An interesting question is to ask anyone involved with glass or buildings what glass is used for. You get all sorts of answers, but it is usually only the brighter architects who hit on the only correct answer at an early stage. It lets light through, either to spread illumination into and through buildings or to provide visual amenity. If you do not want or need the transparency or translucency there are usually better materials than glass to fill the hole or make the structure.

Rationality or mystique

It is difficult to believe that a material which has been known about and used for around 5000 years and has been used in building construction for about 2000 years is only just starting to encroach into the engineering mainstream. Before around 1990, engineers were taught almost nothing about glass and there are still very few courses that do more than touch upon it. Despite glass taking up a larger and larger proportion of the building façade, once the techniques for making much more lightweight structures were developed, it was always delegated to the glass specialists to sort out an appropriate thickness and glass type to provide any glass components.

The reason for this was a reluctance on both sides: the glass manufacturers and suppliers on the one hand not wanting to give out too much information, engineers on the other hand preferring not to get too involved with this awkward material. In one sense the reluctance of the glass manufacturers was simply a continuation of the ancient traditions around glass manufacturing – because it was so difficult to process, those who knew how to do it could keep the secrets to themselves. They had many years of history using certain types/sizes/thicknesses of glass for specific applications. While they knew how to solve a problem, they could not explain to an engineer how they got the answer, particularly since the solutions to different problems appeared to bear no obvious relationship to each other. In order to find out why this historical difficulty arose, it is necessary to know more about the material glass.

Definition of glass

When most people think of glass, they immediately think in terms of glass as used in windows or glass as used for containers. Although these are made of a type of glass, the term glass covers a far wider range of materials. At its broadest the term applies to any material that has been cooled from a liquid into a solid while maintaining the molecular arrangement it had when in a liquid, i.e. it may have short-range order, but on a large scale it is random or amorphous. In theory, all materials that can be turned into a liquid state can be formed into a glass state, but many materials crystallise so readily when they solidify that it is extremely difficult (and in many cases practically impossible) to solidify them sufficiently quickly that they do not form crystals. This is the case, for example, with most metals, and only recently have some metal alloys

been discovered that can be solidified into a glass state, with some interesting effects on the properties of the metal glass compared with its normal crystalline state.

In terms of this book, the meaning of the term glass is restricted to one particular series of materials based on silica (silicon IV oxide, or silicone dioxide, SiO_2). These materials are what we commonly refer to as glass, the silica being adulterated with various additional materials to form specific glass types. An example of this is lead crystal glass (which despite the name is not crystalline in structure) where the silica base material contains a large proportion of lead compounds. Lead crystal glass is rarely used to form glass for use in buildings (only for windows shielding against nuclear radiation or X-rays), being a relatively expensive material with high refractive index which makes it ideal for decorative items.

By far the commonest form of glass used in buildings is generically called soda-lime silicate glass (see **Figure 2** in Chapter 67). There are a few other specialist glass types also used in buildings, mainly to give some fire-resistant properties. Unless otherwise stated, the term glass, when used in this book, will mean soda-lime silicate glass.

Sustainability and recycling

In principle, glass is very good in terms of sustainability and recycling.

In terms of sustainability, it consists largely of silicate materials, which are the among the commonest materials found on earth. Most rocks are silicate based. It will not be possible to ever run out of the raw materials for making glass. However, its manufacturing treatment is to melt the raw materials and resolidify them in a usable form, which requires energy. It is only the availability of suitable energy sources which limits the sustainability of glass.

When glass is formed into a usable object, there are no major chemical changes in the materials. When the usable object needs to be disposed of, it can simply be remelted and re-formed into another usable object – indefinitely. The amount of energy required to remelt glass is considerably less than the amount of energy needed to melt the raw materials used to make it.

Even as landfill, glass is simply returning to its origins as materials from the ground; it is a benign waste material in this sense.

In practice, the picture is not quite so good.

Since the purpose of glass in a building is light transmission, mostly with vision, most silicate rocks are unsuitable; they contain too many contaminants which would make the glass product opaque. Only relatively pure silica, as rock crystal or silica sand, makes clear (colourless) glass. In terms of sustainability this is not a significant problem, as there are more than sufficient sources to last several hundred years – far longer than known energy sources will be available.

Recycling is also not quite as good as first appears. While the general statement about glass being indefinitely recyclable is correct, the recycled products gradually become contaminated, so the glass ends up being a dull green or brown tint, unsuitable for glass in buildings, although it may be practical for use as glass for bottles. The companies manufacturing basic window glass recycle their own waste glass (as cullet – broken glass) and mix it in with the raw materials. They will also take cullet obtained from their customers' waste, provided this is clean and adequately sorted by colour.

Those building glass products which have been subjected to secondary processing may also be more difficult to recycle. In laminated glass, the plastics interlayers are an obvious contaminant. While crushing can break off a proportion of the glass into usable cullet, the interlayer material is not recyclable because of the glass particles which will remain adhered to it. Coated glass can be recycled, but not as clear glass for buildings. Even though the coating layer is extremely thin compared with the glass substrate, it may contain materials which will result in the recycled glass being slightly tinted. Enamelled glass has a coloured glass surface coating fired into it and it may not be suitable for recycling into glass products, as the resulting glass may be opaque.

When waste glass is unsuitable for recycling as glass products, it is a material that is relatively inert, so crushed glass or glass powder can be used as an inert fill for other materials.

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Glass and Glazing Federation http://www.ggf.co.uk/

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Society of Glass Technology http://www.sgt.org/cgi-bin/open.cgi?page=index

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Chapter 67

What is glass?

John Colvin JCGC Limited, UK

The very nature of glass is confusing. It has been described as a solid, or as a liquid, or as a supercooled liquid. The misuse of terms and misunderstanding of the material has given rise to many erroneous beliefs about how glass performs. It helps in designing glass for buildings to know how its nature makes the design approach different from other materials.

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Silicate glass

Crystalline silica, i.e. sand or quartz, is made up of macro molecules (giant molecules) of silicon dioxide (**Figure 1**). The strong single covalent bonds between the atoms join millions of silicon and oxygen atoms in a tetrahedral formation in a giant crystalline structure similar to that in a diamond. The structure is extremely stable because the covalent bonds are very strong, so it has a very high melting point, well over 1200°C.

When silica is heated sufficiently, the covalent bonds break open and continually dissociate and reform, so the particles become free to move and the silica melts. As it does so, the structure becomes less organised, distorting the well-defined tetrahedral formation. This process of dissociating and reforming continues as the molten glass flows, which, unlike water, makes the molten material very viscous even up to extremely high temperatures. While it is possible to melt and form products from pure silica (quartz glass), the high temperatures required make the process costly and it is usually restricted to specialised applications, e.g. chemical glassware.

The addition of other materials which do not form covalent bonds in the molten product breaks up the structure to an extent and makes the molten glass more fluid. Sodium ions (Na^+) and calcium ions (Ca^{2+}) both perform this role and their purpose in glass is as fluxing agents, making the glass reasonably fluid and workable at much lower temperatures than pure silica.

The way in which the molten glass forms covalent bonds between atoms has to be reversed on cooling if the glass is to solidify with a crystalline structure. The strength of the covalent bonds is so effective that the molecular rearrangement required to form the crystalline structure proceeds relatively slowly in silica-based materials and they form a glass on solidifying much more readily than they form a crystalline solid. It is actually quite difficult to get a silica glass to devitrify as the strong covalent bonds need to be broken and reformed.

Figure 2 gives a representation of soda lime silicate glass molecular structure. In reality it is much more complex than the visualisation, as the structure is three-dimensional.

Glass as a material

Glass is a liquid that has cooled to a solid state without crystallising. It is sometimes described as a supercooled liquid, which it is not. A supercooled liquid is a liquid at a temperature below which it would normally solidify. Glass at some stage in its formation is a supercooled liquid, but when it has solidified it is not, it is a solid with an amorphous (i.e. random or non-crystalline) structure. The use of the term supercooled liquid is seriously misleading in that it promotes the idea of flow, but the particles in a glass are not free to move. In fact glass is far too rigid to flow at normal temperatures, within timescales known to man. The fact that many early glass objects are distorted has been used as an argument to support the idea of glass being a liquid which flows. In fact glass is very difficult to manipulate and perfection was extremely costly, so if a glass object was usable it was left imperfect.

On an atomic scale, window glass is basically a network of silicon–oxygen–silicon atoms joined by covalent bonds, this network being randomly modified by the presence of calcium ions and sodium ions (**Figure 2**). The arrangement of these atoms is random, just as it would be in a liquid. The

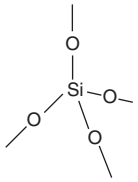

Figure 1 Silica

randomness has been frozen in and it is not orderly or regular like the molecules in a crystal of sugar, salt or ice. Because of this network arrangement, the glass is completely non-ductile and it is this which sets it apart from most other materials.

Stress and strain

When a stress is applied to any material, it deforms through stretching of the interatomic and intermolecular bonds. The amount of the deformation, or strain, depends on the arrangement of the particles and at lower stress levels tends to be linearly related to the stress (**Figure 3**).

At higher levels of stress, most materials deform plastically or yield – that is, they rearrange the particles in the structure, and crystals slide past each other. They can thus often accommodate large strains without failure (**Figure 4**), although they may be permanently deformed.

The structure of glass cannot accommodate this plastic deformation, because any rearrangement of the relationship between the atoms would require the chemical bonds to be broken, so the stress–strain curve for glass (**Figure 5**) shows perfect linearity right up to failure. This is almost unique among common materials.

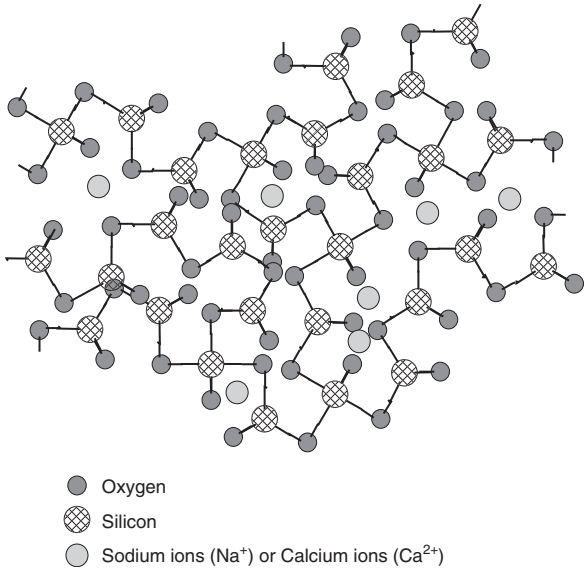

Figure 2 Structure of glass

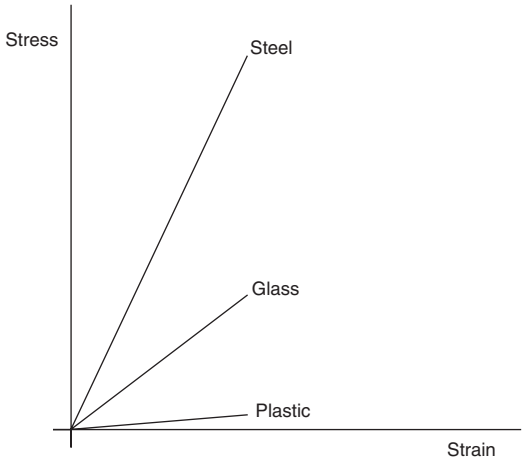

Figure 3 Linear strain

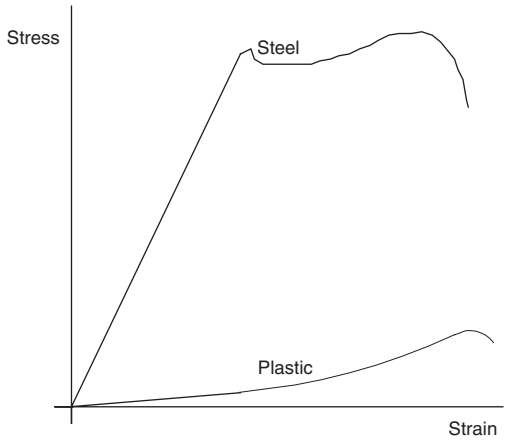

Figure 4 Non-linear strain

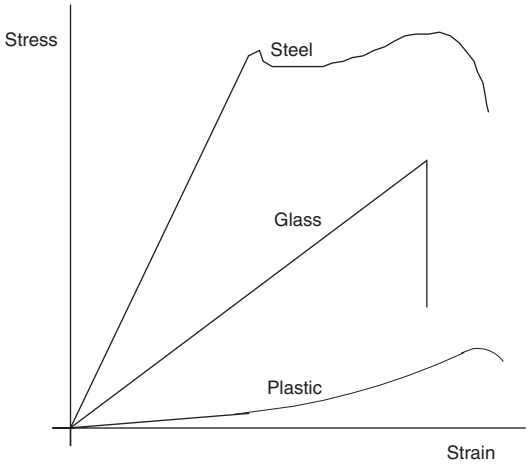

Figure 5 Glass is linear-elastic to failure

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Homogeneity and isotropy

There are two other terms which can be used to describe glass. As a material it is homogeneous. This means that it does not contain any parts with different properties. Concrete is not homogeneous because it contains lumps of different materials: aggregate and sand. Wood is not homogeneous because it contains fibres. Metals are homogeneous until they are viewed at microscopic levels, when it becomes apparent that they consist of adjunct crystals – in alloys these crystals may also be of different materials. Glass is homogeneous at any scale down to atomic dimensions.

Glass is also isotropic. Its properties are independent of direction. Concrete and metals are also isotropic but timber is not, because the fibres run along the grain in one direction only.

The effect of the glass structure on its properties

Several important conclusions about the physical properties of window glass can be deduced from the stress–strain curve.

- There is no creep. When glass is put under stress it deforms to a certain extent and then stops. There is no internal rearrangement of the structure, so there can be no further deformation with time, such as can occur when materials are subjected to plastic deformation.
- Glass is not subject to cyclic fatigue. The perfect linearity of the stress–strain curve means that reversals of stress perform no work on the material and so no damage or work hardening can occur.
- Glass does not suffer from permanent deformation. When a stress or load is removed from a piece of glass, the piece of glass returns exactly to its original shape.
- The absence of any plastic deformation leads to:
  (i) Susceptibility to local overstressing due to its inability to yield locally. This is particularly important for glass fixings, whether conventional frames or bolted connections. Any form of bearing load should be transmitted over as large an area as possible, by a material which will provide the yield or plasticity which glass does not have. Most glaziers are unaware that the primary purpose of the glazing materials – the gaskets, sealants, setting blocks, etc. – is to avoid contact between the glass and any adjacent hard materials and spread out any concentrated forces. Stress concentrations should either be avoided or detailed calculations or tests performed to ensure they are not too high. It cannot be over-emphasised that for glass to have any significant load-bearing capability the most important aspect of design is how it is fixed.
  (ii) High notch sensitivity, which leaves glass vulnerable to flaws. In practical terms, a steel structure can accommodate local stresses and redistribute them by small plastic deformations. The whole structure is very 'forgiving'. Glass cannot do this. It deforms elastically up to the point where the inter-atomic bonds break and a crack forms.
  (iii) Brittle failure. Because glass is homogeneous on any scale down to atomic distances, there are no natural crack stoppers or deflectors. Add this to its high notch sensitivity resulting from a complete absence of any plastic behaviour and it becomes obvious that a crack, once formed, will propagate through the glass at even relatively low levels of stress. Catastrophic failure following the formation of an initial crack is very likely to occur.
  (iv) Low impact resistance. The impact resistance of any material depends on the energy needed to cause fracture. This is strongly related to the area under the stress–strain curve. In the absence of any creep, yield or plastic behaviour, the area under the stress–strain curve for glass is very low, which means that glass is highly susceptible to breakage under impact – as most boys playing ball games discover sooner or later.

Tensile strength levels

The theoretical tensile strength of a piece of silicate glass should (on chemical bond strength calculations) be around $20\,000\ \mathrm{N/mm^2}$.

On freshly drawn silicate glass fibres, tensile stresses of up to $5000\ \mathrm{N/mm^2}$ have been measured and, even when incorporated into a resin to form glass-reinforced plastic, glass fibres have a usable stress of about $1200\ \mathrm{N/mm^2}$.

In practice, the amount of stress needed to break window glass or other forms of bulk glass is very much less than one would expect from considering the forces needed to break the inter-atomic bonds. Window glass commonly fails at stress levels less than $100\ \mathrm{N/mm^2}$ and the origin of fracture is always at the surface.

There must be some form of concentration factor at work.

Surface flaws

In the 1920s A. A. Griffith was working on aspects of aeroplane engines. He investigated the theoretical aspects of stress concentrations in cracks, using glass to experiment with, and arrived at the conclusion that it was relatively easy to develop concentration factors in the order of 200 to 300 at the sharp end of cracks (Griffith, 1920). It is not a great leap of imagination to suggest this as a hypothesis for the low strength of glass in relation to its theoretical strength, with the surface of glass being liberally sprinkled with 'Griffith flaws', although these may not be visible, even with microscopic examination.

The only difficulty with this hypothesis is that no one actually knows what a Griffith flaw in glass is.

The presence of surface flaws, random in size and orientation, leads to some measurable effects. The major one,

which is relatively easy to investigate, is that smaller pieces of glass, when tested, should break on average at a higher strength than larger pieces of glass. This is simply because the probability of finding a more serious flaw in a highly stressed region of the glass is greater when the glass piece is larger. This effect is well known and is mentioned in the European Standard for testing window glass, BS EN 1288-1: 2000. The corollary to this is that if the highly stressed region in a piece of glass is only relatively small, then the glass is less likely to break than if the highly stressed region covers a large area of the glass. This gives rise to the concept of effective strength, which is a statistical determination related to the probability of finding a more serious flaw in the region of high stress. The requirement for a sufficiently serious flaw in relation to the applied stress means that glass does not necessarily break at the position of maximum stress – it breaks at the position where a flaw and the stress combine to exceed the capability of the glass to resist it. This statistical nature of glass strength does not sit well with a traditional engineering approach to design – most engineers are very uncomfortable with the concept.

Scratches, shells, chips and vents

While Griffith flaws are present on the surface of any window glass, the strength of glass is also modified by the presence of major (visible) defects. These can form the origin of cracks, when the glass is put under stress, since the glass is unable to accommodate the local stress concentrations caused by the defects.

The presence of these major defects is one reason why the edge of a piece of glass is usually weaker than the surface. It is much more prone to damage from accidental contact with the surroundings than is the surface. The method of cutting glass can also lead to the presence of some defects in the edge.

Glass and water

The surface of glass is hydrophilic, i.e. it attracts water molecules which stick to the surface using intermolecular forces. Such is the attraction between the two materials that glass needs to be heated up to around 200°C to drive off all the surface water. Even when it is apparently dry, glass has a surface layer of water molecules on it.

The Si–O bonds are susceptible to attack by water, or, more specifically, the small quantity of hydroxide ions (OH^-) in it. If under strain, the bonds can be broken apart and re-form with links to other atoms in an arrangement where the strain is lower.

Water also leaches out the sodium and calcium ions (Na^+ and Ca^{2+}). In doing so, it can form an alkaline solution which contains much higher concentrations of hydroxide ions (OH^-), with a much greater capability for rearranging the glass structure. This is particularly the case if there is liquid water present in small quantities, such as may occur if water is trapped between the layers of glass in a stack. The attack can result in a visible effect. Initially a stain appears on the glass surface, with the appearance similar to an oil film on water. If the attack is allowed to continue, the surface becomes rough, as if it has been etched. In a stack of glass, the adjacent panes can become welded together by this effect, as the chemical bonds are broken and re-form between the two panes where they are in intimate contact.

Static fatigue

The effect of water on the chemical bonds leads to an effect which has become known as static fatigue, but which should more correctly be termed stress corrosion. If the glass surface is under tension, the water which is inevitably present will break the chemical bonds which are under most strain. This will open up any flaw a little bit more, increasing the stress concentration factor and placing higher strain on more of the chemical bonds. The process will continue until the combination of flaw intensity and applied stress combine to create a fast progressing crack and catastrophic failure. This process has been investigated by many researchers, e.g. Charles (1958a, b, c), and Mould and Southwick (1959a, b, 1960). The generally agreed relationship between strength and load duration takes the form

$$\sigma T^{1/n} = \text{constant}$$

where

σ is stress
T is the duration of the stress
n is a constant.

This relationship states that if the duration of stress is increased, the stress at which the glass will break will be decreased.

The value of the constant, n, is the important part of this relationship. Various researchers have obtained values between 12 and 20 for n, depending on conditions of test and composition of glass. A value of 16, in the middle of this range, is generally agreed to give the appropriate relationship for soda lime silicate glass.

One apparent consequence of this formula is that it suggests that at extremely long times, the allowable stress must decrease to a negligible level. In practice this does not occur, and the general formula no longer applies below a certain value of stress. There is a lower limit to the strength of glass, below which it will not fail. The exact level has proved a taxing problem and has not been identified. Various researchers have suggested values between 25% and 40% of the 'short-term strength'.

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Mould (1960), and Mould and Southwick (1959a, b) also investigated the effect of age on scratch damage and found that the strength of the glass increases noticeably if the glass is left for a period of time between creating the scratches and testing its strength. It can be hypothesised that this phenomenon of healing is also related to the way water adjusts the glass structure, breaking apart any strained bonds, which then re-form in a less strained arrangement. It does not then take a great leap of imagination to consider the two processes – stress corrosion and healing – occurring simultaneously. Only if the stress corrosion is occurring at a higher rate than the healing process will the glass eventually break due to the applied stress. For soda lime silicate glass, this base stress level is around 7–10 N/mm^2.

Glass strength testing

Material strength is usually assessed by taking specimens of the material, supported or held in a particular manner and subjecting them to steadily increasing loads applied in a particular way. The stress or load resistance at breakage of a number of specimens can then be statistically analysed to determine mean strength, characteristic strength, etc.

The problem with testing the tensile strength of glass is that it varies considerably depending on:

- the size of the test specimens
- the rate of applying the load
- the distribution of stress in the specimen under load
- whether significant levels of stress are developed at the edges of the specimens.

It is not difficult to get results for the mean strength of window glass which vary by a factor of 2 or 3 depending on the test procedure adopted.

On top of this, the variation of strength between nominally identical pieces of glass, due to the wide range of Griffith flaws which may be present, can be very considerable. In testing 20 nominally identical specimens, using an identical test procedure, the strongest specimen is very likely to be more than 3 times the strength of the weakest specimen.

Historically, before the reasons for the behaviour of glass was better understood, this led to experts with different backgrounds coming up with widely different views as to what values of stress should be used for glass design. Also, because the behaviour was not properly understood, different tests were used to develop design values for different applications. The way in which these values related to the values for other applications could not easily be explained, so the determination of an appropriate glass type and thickness for a particular application became the province of the glass manufactures, using their expertise related to previous successful determinations in similar applications.

Compressive strength

When glass is put into compression, there is no possibility of the surface flaws having a major influence on the strength levels. The theoretical strength of around 20 000 N/mm^2 is a real possibility. In practice, the true value of the compressive strength of glass has never been measured. Any test designed to measure compressive stress has had side-effects giving rise to tensile stresses and it is these which cause the glass to fail. The compressive strength of glass is over 1000 N/mm^2, which means that in practice glass never breaks from compressive stresses.

This does not mean that compressive forces can be ignored. There are always tensile stresses generated in association with compressive forces so the compressive forces, particularly bearing forces, must be controlled so these tensile stresses are limited.

Toughened glass

Compression effects

Since the Griffith flaws and other types of surface or edge flaws only come into action when a tensile stress opens up the flaw, it stands to reason that any method of putting the glass surface into permanent compression would be advantageous. An applied tensile stress would have to overcome this built-in compression before it began to open up a flaw and the glass would be able to resist much higher loads.

Three such products are available. The prestress can be developed thermally, giving products usually called:

- toughened glass (although it is neither harder nor stiffer than untreated glass, just stronger), also known as fully tempered glass, and
- heat-strengthened glass, also known as semi-tempered glass.

It is also possible to generate compressive stress at the surface by chemical modification, the product being generally known as chemically strengthened glass.

Thermal toughening

The mechanism of the toughening process usually used for architectural glass and automotive glass products is based upon slight differences in the contraction of the glass when it solidifies, depending upon the rate at which it is cooled.

As mentioned earlier, the structural rearrangement of the molten glass to form a crystalline structure on solidifying proceeds only slowly, but it is happening. If different parts of the glass cool at different rates, then there will be differences in the amount of restructuring.

The crystalline structure occupies less volume than the random structure, so those parts of the glass which solidify more slowly end up shrinking more than those parts which are solidified more quickly.

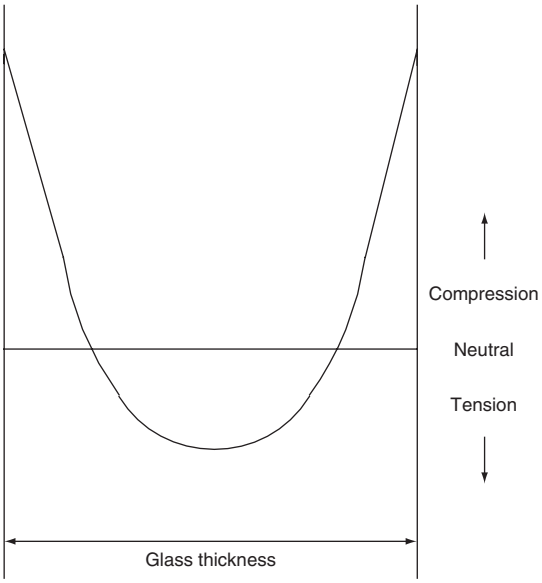

Figure 6 Thermal toughening stress profile

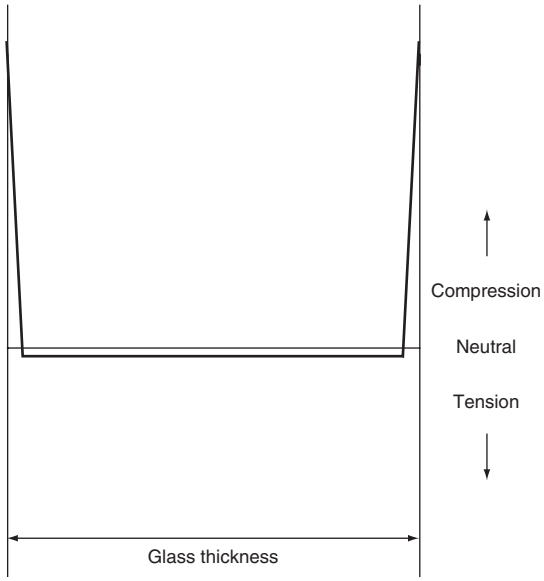

Figure 7 Chemical strengthening stress profile

The thermal toughening process heats the piece of glass to a temperature above its softening point and then rapidly chills the surfaces, commonly by means of air jets. The surface layers of the glass solidify while the central region of the glass is still at a higher temperature.

As this central portion cools more slowly and solidifies, it contracts relative to the surface layers, putting the surface layer into a compressive state and the central region into tension. Using air quenching, this results in an approximately parabolic stress distribution through the glass (**Figure 6**).

For soda lime silicate glass, the surface compressive stress developed in toughened glass (e.g. to BS EN 12150) is typically around 80 to 120 N/mm^2, with a corresponding tensile stress around half this value in the centre of the thickness.

The level of prestressing is controlled by the rate of quenching. Heat-strengthened glass is manufactured with a lower surface compressive stress, typically 30 to 50 N/mm^2 for heat-strengthened glass to BS EN 1863.

Chemical strengthening

It is possible to use the leaching process (whereby the metal ions can be induced to leach out of the glass as discussed in the section on glass and water) to advantage by using materials in which to soak the glass.

If glass is soaked in molten salts containing larger metal ions, there will be an exchange of the metal ions between the glass and the molten salt. The metal ions in the glass (mainly the sodium, Na^+) diffuse out of the surface layers of the glass and the metal ions in the molten salt (usually potassium, K^+) diffuse in to replace them. Because the potassium ions are larger than the sodium ions, they force the surface layer of the glass into compression.

Glass is not very permeable, so this diffusion exchange is limited to a depth of less than 0.25 mm, typically 0.1 mm.

Very high surface compressive stresses, in the region of 300 to 400 N/mm^2 can be developed by this process. The stress profile developed through the thickness of the glass is shown in **Figure 7**.

Fracture

Fracture origin

Cracks in glass always start at a particular point, called the origin. As previously described, the crack starts at position where the tensile stress combines with a flaw in the glass to cause a crack to propagate.

The crack starts at a relatively low speed and accelerates due to the presence of the stress causing it. Around the origin, the fracture surface starts out relatively smooth, resulting in a visible feature called a mirror (see **Figure 8**). As the crack front velocity increases, the crack front becomes unstable and the fracture surface starts to become rough. This is termed fine hackle. The distance of the fine hackle from the point of origin is related to the stress and there is an approximate relationship between the stress which caused the fracture and the mirror radius:

$$\sigma_f = \frac{1.8}{\sqrt{r}}$$

where

σ_f is the breakage stress (N/mm^2),
r is the mirror radius (m).

If the stress is relatively low (and has probably resulted in a crack due to the static fatigue effect) then the crack may

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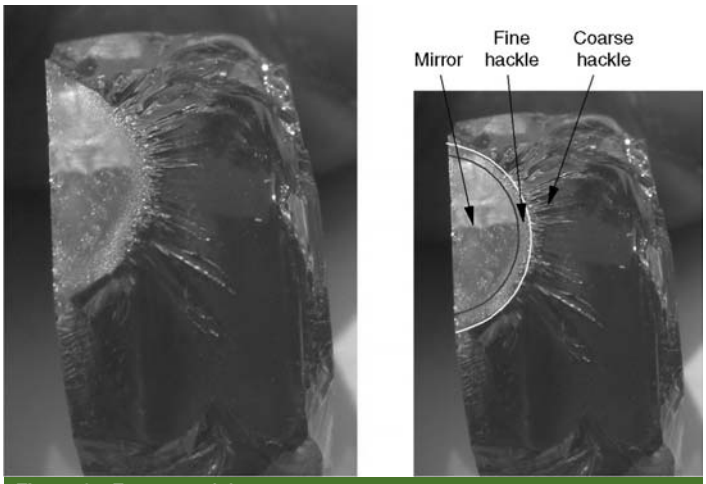

Figure 8 Fracture origin

never attain a velocity sufficient to cause fine hackle. The result is that there is apparently no mirror at the origin. This is a misinterpretation – there is a mirror, but the edges are undefined. This is commonly observed when thermal stress is the cause of fracture.

After the region of fine hackle, the presence of stress will continue to increase the velocity of the crack front resulting in a much rougher fracture surface (coarse hackle). Any subsequent increase in the velocity of the crack front can result in the crack forking. After a crack has forked, the crack front in each branch has a reduced velocity. If there is still sufficient stress present, then the velocity of the crack fronts will increase again and possibly fork again. The number and proximity of forks in the crack gives an idea of the amount of stress present, giving rise to the notion of 'high energy' (e.g. **Figure 9**) or 'low energy' fracture.

Figure 9 'High energy' thermal stress fracture

Wolner lines

As a crack is propagated through the glass, the velocity of the crack front is not constant, it actually proceeds in fits and starts. Each time it stops, it leaves an outline of its position in on the fracture surface. These marks are called Wolner lines. Somewhat counter-intuitively, the crack stops more frequently if its crack front velocity is higher, so the frequency of Wolner lines gives an indication of the overall crack front velocity.

The crack front, and hence the Wolner lines it produces, has a generally parabolic shape with the parabola pointing in the direction of crack propagation (**Figure 10**). If the stress driving the crack is symmetrical through the glass thickness, then the Wolner lines will also be symmetrical. Offset Wolner lines (**Figure 10**) indicate that the stress was due to bending of the glass.

Retracing the Wolner lines back along the fracture surface will act as a guide to find the position of the fracture origin.

Crack forking

While there is a significant amount of tensile stress at the crack front, the overall crack velocity will increase. At higher speeds, the crack front becomes unstable and at around 1500 m/s the crack will fork, giving two crack fronts travelling at a lower velocity. If the tensile stress is still present, these cracks will also speed up and fork in turn.

The amount of forking present in a glass fracture pattern gives a qualitative indication of the 'energy' or stress which caused the cracking. Single cracks indicate low stress or energy; multiple cracks indicate high stress or energy.

Fracture of toughened glass

If a crack is driven through the compressive layer of the toughened glass into the central tensile region, there is enough tensile stress available to make the crack propagate violently through the glass, branching as it goes, to form the

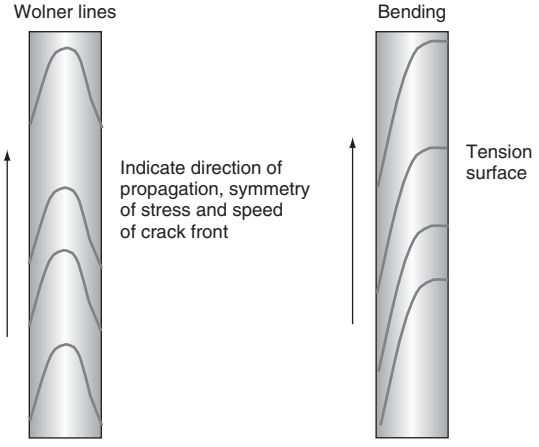

Figure 10 Wolner lines

characteristic 'dice' pattern of broken toughened glass – the most familiar of which are perhaps are the car side screen or vandalised bus stop glass.

The stress driving the crack fronts in toughened glass is the central tensile stress. The higher this is, the more frequently will the crack fork. This has led to the quality assurance (QA) technique of performing a particle count (the number of individual dice in a 50 mm square of the glass) in order to check that the glass has been adequately toughened. A central tensile stress of 45 N/mm^2 (related to a surface compressive stress of 90 N/mm^2) causes forking approximately every 6 mm, generating a particle count of around 60. BS EN 12150 requires a minimum particle count of 40 for most thicknesses of toughened glass.

If the origin of the fracture in toughened glass is not at the edge, there are commonly two larger fragments generated. This is because the fracture starts in two opposite directions, giving the fragments at the origin one side which is approximately twice as long as the average side length for the dice. The two larger fragments are often described as a butterfly, as that is their appearance if the glass does not fall apart after breakage (e.g. if in a laminated glass). **Figure 11** shows how a fracture propagates to form a butterfly.

There is a common misconception that the presence of a butterfly pattern implies one particular cause of failure. In fact a butterfly will form under any cause of failure except high energy impact. To determine the cause of failure, the fracture origin on either part of the butterfly needs to be inspected – the fracture shown in **Figure 11**, for example, was caused by a surface impact from a small hard object.

Fracture of heat-strengthened glass and chemically strengthened glass

Both of these products have a surface compressive stress and an internal tensile stress. However, unlike toughened glass, the internal tensile stresses in heat-strengthened glass conforming to BS EN 1863 and chemically strengthened glass conforming to BS EN 12337 are too low to achieve a velocity of the crack front sufficiently high to make it fork. Both types of glass break in a manner similar to annealed glass.

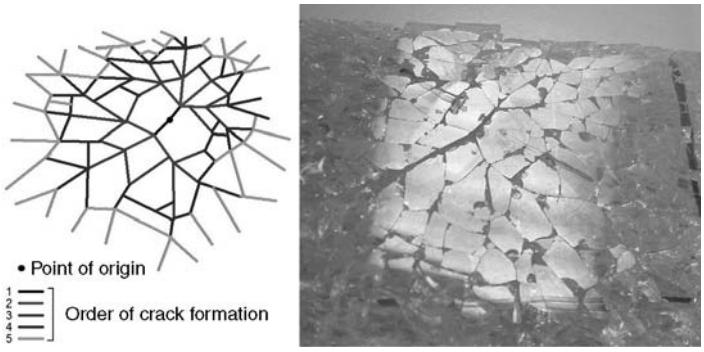

Figure 11 Formation of a butterfly

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Chapter 68

Production techniques – a brief history

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Man has made glass products, decorative items and containers, for more than 5000 years. The use of glass products in construction is more recent – a mere 2000 years. Glass production is very simple in principle, but causes problems in practice which, only in the last 50 years, have been resolved to give cheap, optically accurate, building products.

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Raw materials, melting and annealing

The general process for forming the molten glass is similar for all products and historical periods.

The main component is silica, usually taken straight from relatively pure silica sand deposits in the ground, with no further processing than washing.

As previously described in Chapter 67, soda (potassium oxide can also be used) and lime are added as fluxing agents. Soda is the better fluxing agent, but too much soda makes the glass susceptible to weathering damage. These components are added in the form of carbonates. Lime comes in the form of limestone or dolomite (which contains a proportion of magnesium carbonate), taken straight from quarries with no further process than washing. Soda is added as sodium carbonate (also known as soda ash and washing soda). In modern times sodium carbonate is produced as one of the by-products of manufacturing chlorine, but in earlier times soda ash and potash would have been used, taken from the ashes of fires.

The addition of soda and lime as carbonates produces small bubbles of carbon dioxide in the molten glass. Since molten glass is very viscous, these bubbles take a long time to rise to the surface and escape. In small batch mixes this is not a significant problem, but with modern continuous production processes it is a serious restriction. To speed up this process, sodium sulfate is added to the mix. This makes bubbles of sulphur dioxide in the molten glass. While at first glance it may seem contrary to add even more bubbles, sulphur dioxide forms much larger bubbles into which the carbon dioxide bubbles merge. The larger bubbles rise much more quickly, speeding up the bubble clearing process. A large glass melt takes about three to four days to clear.

The final major component raw material for making glass is cullet – broken glass. Up to around 15–20% of the raw material in the mix can be cullet. The cullet melts at a much lower temperature than the other components and its presence speeds up the melting process, particularly important for modern glass production processes.

For modern soda-lime silicate glass for buildings (and transport), the proportions of the raw materials are assembled in order to produce glass with a composition in the range shown in **Table 1**, taken from EN 572.

The raw materials are mixed and ground to form a powder called a frit. The frit is then melted to form the molten glass. This used to be done as a single melt process, but modern glass products are made by continuous processes whereby the frit is added continuously at one end of a tank and the melted and refined glass is drawn off at the other end of the melting tank. The melting temperature is typically around 1600°C. Once the glass is refined (most of the bubbles have come out and any contaminants have dissolved into the glass), it is allowed to cool until it forms a very viscous liquid (around 1000°C to 1100°C for soda-lime silicate glass, lower for lead glass), at which point it can be further processed.

After being solidified in the desired form, all glass needs to be annealed. The annealing process is slowly cooling the glass from a temperature above its softening point (typically around 550°C for soda-lime-silicate glass) so that

Silicon dioxide (SiO_2) silica	69–74%
Calcium oxide (CaO) lime	5–12%
Sodium oxide (Na_2O) soda	12–16%
Magnesium oxide (MgO) magnesia	0–6%
Aluminium oxide (Al_2O_3) alumina	0–3%

In addition to the main components listed, the glass also contains small amounts of other materials which may have been in the raw ingredients, e.g. iron oxides (which give clear glass its typical green tinge), or may have been deliberately added to give the glass colour.

Table 1 Modern soda-lime silicate glass composition

adverse stresses are not built into the glass. The annealing typically takes place in an oven where the glass is allowed to cool down slowly. In modern manufacturing processes which produce a continuous ribbon of glass, the oven is called a lehr, which the solidified ribbon of glass passes through as it gradually cools.

Roman window glass

Although glass was manufactured for thousands of years before the Roman era, it was always coloured glass and formed into decorative items and containers. The Romans were the first manufacturers to make significant quantities of clear (i.e. colourless) glass, particularly for containers, but they also used it in windows. From descriptions, glass in windows appears to have been a status symbol.

Early Roman glass appears to have been poured as a liquid onto a flat slab and then pulled into a rectangular shape while it was still soft. The glass panes formed in this way were relatively thick for their size and were badly distorted. They may have let light through, but vision would have been badly distorted.

Glass casting onto a table was used right up to the nineteenth century to produce particular variations of product. It was effectively made obsolete when techniques for making thick sheet glass were developed.

Cylinder glass

Cylinder glass, also known as munt glass, was manufactured by blowing a bubble of glass, and then twirling the blowing rod like a cheerleader's baton, which stretched the bubble into a long cylinder. The cylinder had both ends removed and was subsequently cracked down its length using a hot bar. Reheating the split cylinder allowed it to be flattened out into a rectangular pane. This method produced glass which was thinner and had much better visual clarity than cast glass.

There is evidence that some late Roman glass was also made using this process. The cylinder process was used right up to the nineteenth century, by which stage the blowing and twirling had been mechanised so that large panes could be manufactured.

Cylinder glass was made obsolete by the development of sheet glass processes.

Crown glass

Crown glass was made by collecting a large blob of molten glass onto the end of a rod and simply spinning the rod on its axis. The centripetal forces on the glass caused it to spread out into a disc. A skilled crown spinner could form a disc around 1.8 m in diameter, but smaller discs were more common. The glass from a crown was always thickest towards the centre of the disc and thinnest near the rim.

On cooling, the glass was broken off the rod and cut into glazing sizes. The characteristic 'bullion' in the centre was either discarded or put back into the glass melt. Occasionally it would be sold cheaply to customers who could not afford to glaze their windows completely with good-quality glass.

Crown glass has given rise to one of the myths about glass – that old glass, because it is a liquid, has sagged, so it is thicker at one edge. This is untrue, the glass was like that when it was made.

Crown glass was made obsolete by the development of the sheet glass process.

Sheet glass

Just as one can draw a trail of syrup by dipping a spoon in it, it is possible to draw a trail of glass by dipping a piece of metal in it. Using a metal circle and by carefully controlling air pressure inside, it was found possible in the mid-nineteenth century to draw cylinders of glass up to 50 ft (16 m) long. These cylinders would then be cut, split and flattened as were the blown cylinders.

It was not until 1913 that flat drawn sheet glass was first manufactured. The drawing was started by using a metal bar. The most effective process then used a series of knurled wheels at the edges of the ribbon of glass to stop it from 'waisting' before it set. The glass was then pulled through an annealing lehr, where the glass ribbon was cooled sufficiently slowly that the glass did not crack and the residual stresses were sufficiently low to allow the glass to be cut. Once this process was started, it could be maintained indefinitely if the melting tank was continually replenished and the annealing conditions kept stable. Sheet glass became the first serious mass-produced glass.

The finished product has two bright (fire finished) surfaces because both were untouched before the glass set and gave the highest visual clarity. Unfortunately the thickness was variable, both across and along the glass ribbon, and the glass gave a visual distortion of objects viewed through it which is all too obvious. The overall visual quality was much superior to earlier cylinder glass and the product was much cheaper than plate glass.

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Sheet glass is still manufactured and is covered by BS EN 572-4.

Plate glass

Plate glass is glass formed into a flat pane by any suitable process, for example table cast glass, cylinder glass or sheet glass, which is subsequently ground and polished on both surfaces to produce a piece of glass which has both surfaces flat and parallel.

The initial grinding to flatten the surfaces is performed using a coarse grit or sand. This is followed by using an abrasive medium which is successively finer (very often this is waste grinding sand which has been regraded for use at a later stage of the process). The final polish is given to the glass surface by using a polishing material such as jeweller's rouge.

Only in the nineteenth century, when the process could be mechanised, did plate glass become relatively common. One manufacturing plant whereby both surfaces can be treated simultaneously on a continuous process, although built in the early twentieth century, is still in use to manufacture polished wired glass.

The development of the float glass process made clear plate glass obsolete.

Rolled glass

Not all glass is required for its visual clarity – indeed there is a need for certain windows to let light through while obscuring vision.

Rolled glass is manufactured to a controlled thickness by passing the soft glass between two rollers (**Figure 1**). The resulting product does not have very good visual clarity, even if the two rollers are smooth faced. A patterned glass can be manufactured by giving one of the rollers a patterned surface.

Although the process was developed in the mid-nineteenth century, modern decorative patterned glass products are made using essentially the same process. Patterned glass is covered by BS EN 572-5.

Wired glass

Wired glass is also made by the rolling process. The first wired glass was made in around 1895 in response to the need for a type of glass which would prevent the easy passage of flames and smoke both inside buildings and between them. Because wired glass has a tendency to remain in position when cracked, it very quickly found another market in the large glazed canopies of railway stations.

There are two basic methods of manufacture.

One manufacturing method uses a process similar to **Figure 1** where a wire mesh is introduced under the top roller, which is ribbed to push the wire into the centre of the glass thickness.

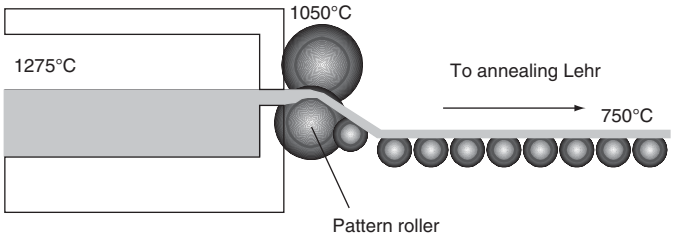

Figure 1 Rolled glass

An alternative process splits the glass feed into two separate channels and converges them one above the other, with the wire fed in between the two glass layers (**Figure 2**). The glass and wire sandwich is then fed between rollers to control the glass thickness.

Both of these manufacturing methods are still in use for manufacturing the product currently known as 'wired patterned glass'.

Polished wired glass is simply wired patterned glass which has been subsequently ground and polished using the plate glass process as described earlier.

Wired glass is now rarely used in roof glazing, although it may be found in older buildings. However its ability to give proven fire resistance at a reasonable cost means that it is still commonly used in fire doors and fire screens.

Wired patterned glass is covered by EN 572-6 and polished wired glass is covered by BS EN 572-3.

Float glass

The idea for float glass was originally thought up in around 1900, but there was no way at the time of producing it economically. Pilkington Brothers Ltd reinvented it in the late 1950s and developed a workable manufacturing process. Float glass is formed by taking the still molten glass, at around 1100°C and drawing it out onto a bed of molten tin (**Figure 3**).

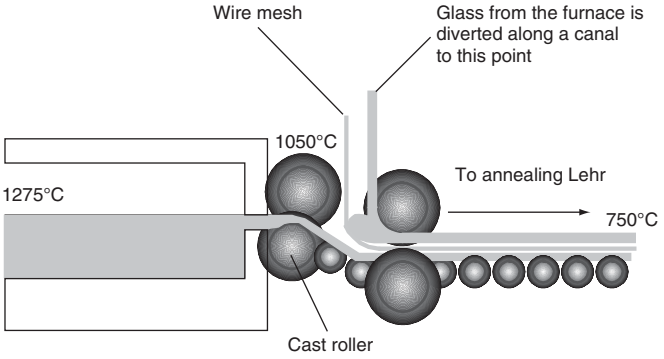

Figure 2 Wired glass process

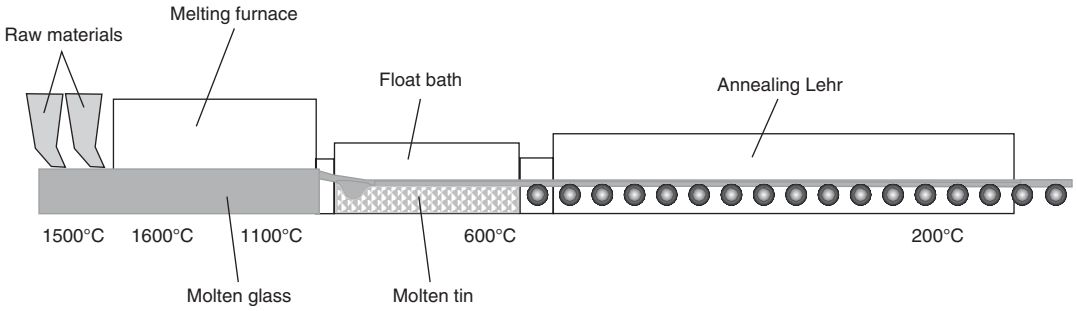

Figure 3 The float glass process

Liquids naturally adopt very flat surfaces, so this process forms a sheet of molten glass which is naturally very flat on both surfaces. As the sheet of glass is drawn across the tin it slowly cools down and solidifies. At the end of the tin bath, at a temperature of around 600°C, the glass is solid but soft. At this stage it is pulled out of the float bath into an annealing lehr, where the glass is cooled slowly to a temperature around 200°C before it is washed and cut into jumbo sizes or smaller stock sizes. The natural thickness of float glass is around 6 mm. By damming it back or pulling it out of the float bath more quickly, thicker or thinner glass can be made. Float glass from 0.5 mm to 25 mm thick is commercially available, although it is not practical to make the full thickness range on one float line. For glass used in buildings, the thinnest float glass commonly used is 3 mm thick.

The huge advantage of having a glass product which could compete with sheet glass on price and rate of production, but with the freedom from distortion that was previously only achieved with costly plate glass, revolutionised glass manufacture and usage. The process is now used to produce almost all building glass.

Float glass is covered by BS EN 572-2.

Multiple glazing

Definitions

Multiple glazing is a general term that covers several different options for glazing. In the modern world, the term brings to mind insulating glass units, products where two or more panes of glass are spaced apart and hermetically sealed around the edges. Prior to the advent of insulating glass units, and also still being used, is secondary glazing, where the panes are glazed or fixed separately. The two types can be used in combination; in Scandinavia the current trend is towards triple-glazed insulating glass units with a fourth pane which is secondary glazing.

Secondary glazing

Multiple glazing tends to be thought of as a relatively recent development, but the earliest known examples date from the eighteenth century. Perhaps more surprising is that the earliest use was not for thermal insulation to keep heat in the building, but by glazing the two panes separately, some distance apart, with the cavities being open between panes which were one above another and also open at the top and bottom. The purpose of this was to employ what is considered a very modern idea of using the 'stack effect' of the warm air in the gap to make it convect and remove hot air out of the open top, drawing cooler air in at the bottom – an effective form of solar control. The main use was for orangeries, in climates with hot summers, in order to prevent the valuable exotic plants from being scorched.

Multiple glazing for buildings used by people was not considered until early in the nineteenth century, when it started to be used in the form of secondary glazing. The purpose of this initially was primarily to eliminate the cold downdraft from single-glazed windows, but it also began to be used to improve sound insulation of windows. It was not until the second half of the twentieth century that the thermal insulation properties of multiple glazing began to be seriously recognised. Secondary glazing was still commonly used for upgrading the thermal insulation and sound insulation of windows until insulating glass units became readily available in the 1980s.

Thermal insulation

Multiple glazing improves thermal insulation by the presence of the cavity between the panes. Thermal insulation is rated by the U value, a measure of the rate at which heat escapes through the building fabric. Single glazing has a U value around 5.7 W/m^2K. Normal double glazing has a U value around 2.5 to 3.5 W/m^2K, depending on its construction.

Heat crosses the cavity by three different processes: conduction, convection and radiation. For a cavity width of 12 mm, in double glazing using normal glass and air filled, the rate of heat transfer is approximately the same for each of the three processes. The amount of heat transferred by conduction is reduced by making the cavity wider. Unfortunately the amount of heat being transferred

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Cavity gas	Optimum cavity width: mm	U value: W/m^{2}K
Air	16	2.70
Argon	16	2.56
Krypton	11	2.51
Xenon	8	2.47

Unit consisting of two panes of 6 mm float glass

Table 2 Effect of gas filling on U value of clear float glass insulating glass units

Cavity gas	Optimum cavity width: mm	U value: W/m^{2}K
Air	16	1.35
Argon	16	1.09
Krypton	11	0.94
Xenon	8	0.85

Unit consisting of one pane of 6 mm float glass and one pane of 6 mm low E glass with emissivity 0.02

Table 3 Effect of gas filling on U value of clear float glass insulating glass units

by convection is reduced by making the cavity narrower. With air filling, there is an optimum cavity width of around 16 mm where the U value is at its lowest. Wider gaps increase the U value only slowly, so secondary glazing, assuming there are no major ventilation gaps into the cavity, can be as effective as insulating glass units.

The balance between conduction and convection can be altered by changing the gas filling in the cavity **(Table 2)**. A common gas filling for improved thermal insulation in insulating glass units is argon. This reduces both conduction and convection to a similar extent, meaning the optimum cavity width is still around 16 mm. Depending on other factors, replacing the air with argon generally reduces the U value of an insulating glass unit by between 0.15 and 0.4 W/m^{2}K. More exotic gas types which have occasionally been used are krypton and xenon, which affect conduction and convection differently, so the optimum cavity widths for these gases are different.

Rather than having a cavity filled with gas, it is theoretically better to have a vacuum. Various companies and research centres have been investigating this possibility for well over 30 years, but the big problem is that a sealed cavity with flat sides simply collapses under atmospheric pressure. While it is relatively easy to find ways of stopping the cavity collapsing, glass is principally used for looking through and no one has yet found a method of doing this that is not visually obtrusive.

Radiation from one pane to the other is direct irrespective of the medium (or lack of it) in between. The amount of heat transferred by radiation is not affected by the cavity width or the type of filling. It is affected by the emissivity of the surfaces.

Coated glass with a coating incorporating silver has surface emissivities between 0.02 and 0.12, depending on the layering of the coating. Using such a coated glass as one pane of an insulating glass unit can largely eliminate the radiation component of heat transfer across the cavity of the insulating glass unit, reducing the U value by 40% or more compared with a unit made only from ordinary glass, which has an emissivity of 0.9.

Because the radiation component is unaffected by the cavity width or the gas filling, the effects can be combined, giving U values which are considerably lower than the equivalent uncoated glass products (**Table 3**).

Insulating glass units

The big disadvantage of secondary glazing is that the gap between the panes is not hermetically sealed. Dust particles get inside the gap and the glazing becomes unsightly. Depending on the environmental conditions, condensation may also form in the gap. This means that the secondary glazing must be openable or removable in order that the gap can be cleaned at intervals.

While there is evidence from the nineteenth century of some attempts to make insulating glass units, these came to nought because the technology was not available to make units that would not get condensation inside shortly after they were manufactured.

The first durable insulating glass units were made in Belgium in the 1930s, by spacing two panes of glass apart with a copper spacer bar soldered between them near the edge (**Figure 4**). A caulking compound was used to protect the soldered joint. A few of the earliest installations still surviving have a high proportion of the original units glazed. Variations on this theme were developed in the 1940s and 1950s using other soft metal spacer bars and either welding or gluing them in position.

Alternative manufacturing methods developed in the 1950s held the two panes apart and melted the glass together around the edges forming an all-glass insulating

Emissivity

Emissivity is a property of a material surface which measures its ability to absorb or emit electromagnetic radiation (light, heat). It is rated on a scale of 0 to 1. The emissivity for any material may be different at different radiation wavelengths. Materials which have a low emissivity at light wavelengths make good mirrors; light reflects off them. Many metals make good mirrors; silver in particular has a very low emissivity, <0.02. Most materials have a high emissivity around 0.9. Coated glass with a surface emissivity less than 0.2 is defined as a low E glass.

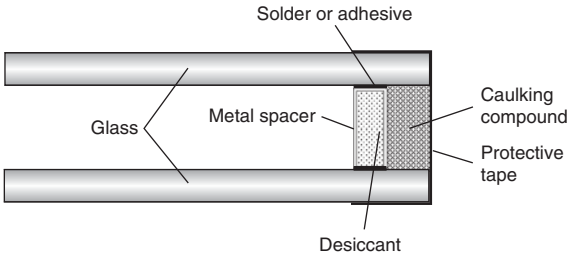

Figure 4 Early types of insulating glass unit edge seal

unit (**Figure 5**). These types of unit had no desiccant and did not need it because glass is impermeable to moisture vapour penetration. Unfortunately, it is impractical, due to the rigidity of the edge seal, to provide all-glass insulating glass units with a cavity width much more than 9 mm and it is not possible to incorporate toughened glass, laminated glass or coated glass.

Most of these early methods of making insulating glass units have disappeared since the process of manufacturing insulating glass units with polysulphide sealant combined with aluminium spacer bar containing a desiccant was developed in the 1960s. This production method needed much lower capital investment and much less labour than any of the previous methods and is also much more versatile. The earliest versions of these polysulphide units were not particularly durable, but the introduction in the 1970s of the modern dual seal unit (**Figure 6**), having a bead of polyisobutylene (commonly known as PIB or butyl) between the aluminium spacer bar and the glass-produced units with much greater durability.

The effectiveness of butyl as the primary seal in a dual seal construction allowed other materials to be used for the secondary seal, such as silicone and polyurethane. The use of silicone, in particular, has the advantage that the edge seal can be exposed to direct sunlight, as all the components are resistant to UV degradation. Insulating glass units with silicone edge seals can be used where the glass is not framed all round.

Other variations on the edge seal are available. The aluminium spacer bar can be replaced by stainless steel or metal-coated plastics, both of which have a lower conductivity than aluminium and this may give a minor improvement to the thermal insulation of insulating glass units.

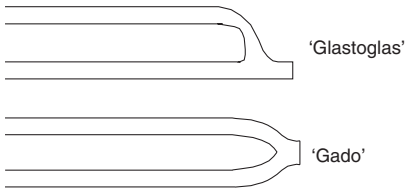

Figure 5 Two types of all-glass edge seal

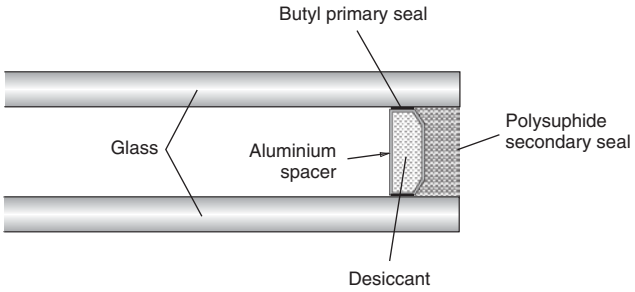

Figure 6 Dual seal construction

A spacer system 'Swiggle' is available which eliminates the spacer bar, being a butyl rectangular section with a steel insert to keep its shape and desiccant incorporated into the material. Thermoplastic spacer (TPS) is a modern version of this idea. Alternatively, using hot melt butyl for the secondary seal means that the butyl primary seal between the spacer bar and the glass is unnecessary. All of these have tendencies to soften if the insulating glass units get hot in service and the edge seals can distort.

The product standard for insulating glass units is BS EN 1279.

Laminated glass

Early laminated glass

Laminated glass was first manufactured commercially in around 1910. It was similar to modern laminated glass in that a type of plastic sheet was used as an interlayer both to fill the gap between two plies of glass and to glue them together.

Because the plastic sheet needed to have two closely matched surfaces, the glass needed to be plate glass. The manufacturing process was also similar to some modern laminated glass in that the sandwich of glass and plastic sheet was warmed up until the plastic melted into intimate contact with the glass.

Early interlayers were based on celluloid, which was highly susceptible to moisture attack and was biodegradable. Although laminated glass was used in the windscreens of early cars, it used to deteriorate rapidly from the edges. The use of laminated glass in car windscreens almost ceased when toughened glass started to be manufactured in the 1930s, because of this durability problem.

Pvb (polyvinyl butyral)

Laminated glass started its comeback with the advent of polyvinyl butyral (pvb) in the 1950s (**Figure 7**). This interlayer material had much superior energy-absorbing properties and much superior durability than celluloid. During the 1950s and 1960s laminated glass started to replace toughened glass as the choice for car windscreens, but it was not until the 1970s that it started to be used in any significant quantities in

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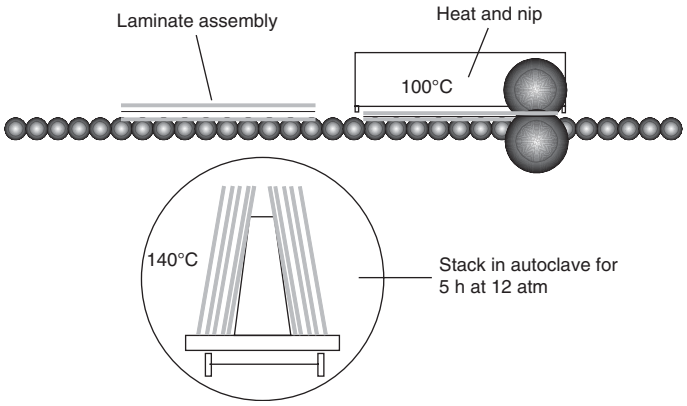

Figure 7 Pvb laminating process

buildings – initially as a more aesthetic alternative to wired glass in roofing applications. By the late 1970s it was also being used for its safety properties, which were underscored when BS 6206 was published in 1981.

Pvb laminated glass is manufactured by layering plies of glass and plies of pvb to form the appropriate laminated glass construction. The commonly available types of laminated glass are three-ply – two plies of glass (usually the same thickness) – and the pvb interlayer.

Pvb is supplied in thickness multiples of 0.38 mm (15 thou in imperial units). Some common thicknesses of laminated glass are 6.4 mm (3|0.38|3), 6.8 mm (3|0.76|3), 8.8 mm (4|0.76|4), 10.8 mm (5|0.76|5), 11.5 mm (5|1.52|5).

(Note: if a laminated glass is described simply by thickness, without stating what glass is to be incorporated in the product, then it will be assumed by the glass supplier that the glass is clear float glass. If the laminated glass is required to be made from other types of glass, then this needs to be explicitly stated.)

The energy-absorbing properties of the pvb interlayer made laminated glass difficult to penetrate deliberately, especially when there were more than three plies of glass and/or thicker interlayers were used. Laminated glass, 70 mm thick with over 10 plies of various thicknesses of glass and pvb interlayers, is capable of stopping an armour piercing rifle bullet. It has become common to use glass resistant to manual attack (BS EN 356) or bullet-resistant glass (BS EN 1063) in glazing where security is required.

Subsequent to pvb, other materials have been used for the interlayers in laminated glass. Some of these are briefly discussed in the following sections.

Polyurethane

Polyurethane has slightly superior properties to pvb in some areas, although it is more expensive. Laminated glass incorporating polyurethane can be made using the same process as pvb. It has the big advantage that it can be used to laminate polycarbonate sheet to glass. A composite laminate containing both glass and polycarbonate can give the same penetration resistance as a laminated glass twice as thick which uses only glass. This gives significant weight savings – particularly useful in transport applications, especially for aeroplane windscreens and train windscreens which need to be capable of resisting considerable impacts. Polyurethane interlayers can also be made using the cast-in-place process (see below). While the casting versions do not have as good post-breakage performance as the sheet versions, they can be used for laminating polycarbonate interlayers.

EVA

EVA is a material available in sheet form which does not need to be put through an autoclave in order to cure it. It has higher strength and stiffness than either pvb or polyurethane, but is more expensive than pvb and cannot be used to laminate polycarbonate like polyurethane.

Cast-in-place interlayers

Laminated glass can also be made using liquid interlayers which subsequently set. The glass plies are spaced apart by an adhesive edge tape, which is opened on one edge and the liquid poured into the gap. The opened edge is subsequently closed and any trapped air is drawn out, leaving the gap between the panes filled with the liquid. The liquid then sets, either chemically by the prior mixing of a catalyst, or by exposure to UV light. Despite the simplicity of the process, it can be used to manufacture large and complex products (**Figure 8**).

The advantages of cast-in-place (CIP) interlayers is that the set-up cost is relatively low and the interlayer material is guaranteed to fill the gap between the panes. The latter

Figure 8 CIP laminated glass manufacture (courtesy of Hansen Glass)

allows the laminating of glass which has undulating surfaces which cannot be laminated using a flat plastic sheet. In the early 1980s, when CIP interlayers were first introduced, it became possible to easily laminate thick toughened glass, which was very difficult using pvb due to the distortion in the glass. It can also be used to laminate any patterned glass.

The disadvantages are that the CIP interlayers have much less capacity for energy absorption and tear resistance than pvb and polyurethane, so that the post-breakage performance of the laminated glass product is nowhere near as good (although it may be adequate for some purposes). It also has developed a reputation for inconsistency and being more susceptible to delamination, although this reputation may not be deserved.

There are two basic types of CIP interlayers. Polymethylmethacrylate (PMMA) is generally formulated to be relatively soft, which gives it excellent sound absorption properties. Polyester is generally formulated to be more rigid than pvb, which gives it certain structural capabilities not available when using pvb or PMMA. Polyurethanes are also available as casting resins for laminating.

Ionomer interlayer

An ionomer with the trade name SentryGlasPlus® is the first interlayer material which has been designed specifically for laminated glass in the areas of general safety, security and structural performance. Pvb, polyurethane and the CIP interlayer materials were all modified versions of materials which had other functions and which happened to find a use as a laminating interlayer.

SGP is much stiffer and stronger than pvb and allows laminated glass to perform mechanically, both as a complete product and in post-breakage performance, in a much superior way to other interlayers. It is, unfortunately, considerably more expensive and more difficult to process than any of the other interlayer materials.

Product standards

All laminated glass products are covered by BS EN ISO 12543 and BS EN 14449.

In addition to the general laminated glass standards, laminated glass may be covered by other standards for particular applications.

In terms of security glass, glazing resistant to manual attack (sometimes referred to as anti-bandit glass) is classified according to BS EN 356. Bullet-resistant glass is classified according to BS EN 1063. Blast-resistant glass is classified according to BS EN 13541.

Toughened glass

There are a few references in medieval alchemists' writings which mention a type of glass which is very strong, but it is not clear whether any was ever made, or whether it was simply an ideal.

The earliest known form of toughened glass is Prince Rupert's drops, which are an interesting, but otherwise useless form of glass. The story goes that a Prince Rupert (who may have been King Charles I's leading cavalry officer – the date is about right) wanted to know what shape raindrops were, so he hit on the idea of making drops of molten glass and dropping them into water to freeze them. The resulting bits of glass have a bulbous head and a long thinning tail (which, incidentally, is not the shape of raindrops). These bits of glass have the remarkable property that the head can be hit extremely hard with a hammer without any breakage, but the tail is fragile and easily broken, whereupon the head explodes into numerous small fragments.

Usable toughened glass was first made in France in about 1870, when small thick pieces were heated to a high temperature and dropped into water to quench them. These were used for sight glasses in pressure vessels. The process did not make glass with good visual quality and it was not possible to make large sizes.

The reason for the properties of Prince Rupert's drops was resolved in the 1920s (see box).

Toughened glass as we know it today (see section on Thermal toughening in Chapter 67) was first manufactured in the 1930s under the trade name 'Securit', by a process which is now called vertical toughening. The sheet of glass was hung from tongs along one edge and was transported into a furnace. The glass was left, hanging from the tongs in the furnace, until it heated through to a temperature between 600°C and 640°C. As soon as it was at the correct temperature, it was removed from the furnace and positioned between two sets of high-pressure air jets which were used to quench it. Glass becomes soft above about 550°C, and the tongs dig into the glass producing characteristic dimples or tong marks along one edge of the glass.

The characteristics of the product were that it was several times stronger than annealed glass and if it broke it disintegrated into very small pieces, called dice, which did

Prince Rupert's drops

When glass is quenched quickly, the surface of the glass goes into compression and the interior is set in tension. For a given rate of quenching, the stress developed is proportional to the thickness of the glass, so the bulbous head develops high surface compressive stress. This makes it very difficult to break apart. On the other hand, the tail is very thin and the surface compressive stress developed is low, so it is relatively easy to break. Once cracks are in the interior tensile stress zone they propagate rapidly. With the high tensile stress in the centre of the bulbous head, the energy released by the fracture is sufficient to blow fragments in all directions.

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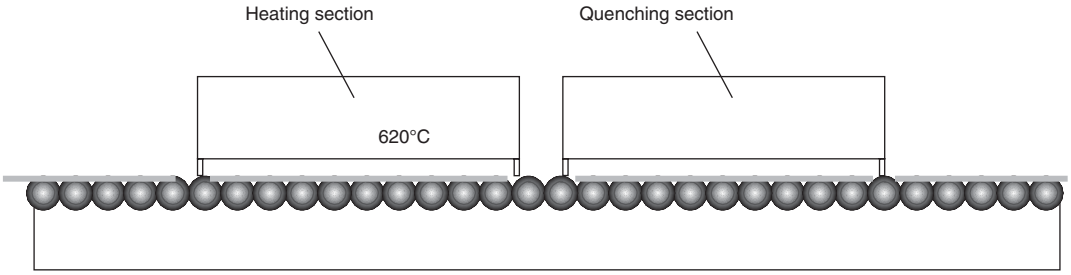

Figure 9 Horizontal roller hearth toughening

not have the lethally sharp edges of broken annealed glass. The availability of a strong and much more robust glass product, which, when it did break, broke 'safely', was an instant success in the car industry and very quickly became the standard glass product used in any form of transport. Its properties were also recognised for glass in buildings and toughened plate glass, 'Armourplate', became used regularly for large panes in prestige buildings.

In about 1970, the process of horizontal roller hearth toughening (**Figure 9**) was developed.

The rollers are made from ceramic materials and have to be very flat. Because the glass becomes relatively soft above 550°C, it has a tendency to drape between the rollers. In order to try and avoid this occurring, the glass is continuously oscillated backwards and forwards on the rollers. Once the glass has reached a temperature between 600°C and 640°C, it is rolled into the quench where, either in a static position or still oscillating backwards and forwards, it is quenched by high-pressure air jets from above and below. The big advantage of the roller hearth furnace is reduced distortion of the glass. The tendency for the glass to bow is more easily controlled and there are no unsightly tong marks. On the down side, the roller hearth furnace causes roller wave to occur and the weight of the glass on the rollers can cause small dimples in the surface.

One of the characteristics of toughened glass was that holes (drilled before toughening) had usable strength, so local fixings could be used, leading to early versions of 'Armourplate' frameless glass doors, which soon led to the use of patch plates to connect several panes, supported by small glass fins in small entrance assemblies.

By the late 1960s, Pilkington had started to develop these into much larger-scale 'Armourplate Assemblies' with properly engineered structural toughened glass fins, using friction grip fin root clamps and splices to support the bolted façade panes. Once the visual obtrusiveness of tong marks was eliminated, the size of these bolted glass assemblies rapidly increased as architects developed a taste for them. The first major use of this construction in the UK was the Willis Faber Dumas building in Ipswich in 1974, where the entire 450 m length of the irregularly shaped perimeter façade, 15 m high, was a suspended glass assembly construction braced by glass fins.

Toughened glass, because of its high and reliable strength, today forms the basis of most glass constructions which are considered structural or semi-structural. It is also used for the majority of glass in transport, except for windscreens, which are laminated glass.

Toughened glass for use in buildings is covered by BS EN 12150. The majority of modern toughened glass is made by the roller hearth method, but some is still manufactured using vertical toughening.

One of the problems with toughened glass is its vulnerability to small inclusions made from nickel sulphide. These inclusions, which are only 50 to 250 microns (0.05–0.25 mm) in diameter are present naturally in some panes and cause the glass to shatter. These were first identified in 1962 and led to the development of two variations of toughened glass, heat-strengthened glass and heat-soaked toughened glass.

Heat-strengthened glass

The high stresses built into toughened glass are what make it vulnerable to nickel sulphide inclusions. The stresses are built in to the glass during quenching and, simply by reducing the air quench flow, a product with lower prestressing can be made. Heat-strengthened glass is very much less susceptible to nickel sulphide breakages than is toughened glass but not completely proof.

Heat-strengthened glass is sufficiently stronger than annealed glass that it is not susceptible to thermal stress breakages in ordinary building use, but it does not have the strength of fully toughened glass and it breaks in a manner which is closer to that of annealed glass, giving large shards with very sharp edges, so it is not a safety glass.

Heat-strengthened glass is covered by BS EN 1863.

Heat-soaked toughened glass

The other way of getting rid of nickel sulphide inclusions is to make any critical ones cause fracture before the glass leaves the factory. Such a product is heat-soaked toughened glass. Until the mid-1990s the mechanism of the heat soaking

process was only partly understood, but an effort by several European glass companies led to a proper understanding and development of the optimal heat soaking process (see section on Toughened glass to BS EN 12150 in Chapter 70) which forms the basis of the European Standard BS EN 14179.

Enamelled glass

Enamelling is the process of applying a glass material to a substrate. This process can be applied to many substrate materials; enamelled baths and cooking items immediately spring to mind. It is also applied to pottery and china, where it is usually called a glaze. It has been applied to a base glass substrate for many centuries. Many of the varieties of coloured glass used for stained glass are a cheap glass substrate with the expensive coloured material painted on and 'flashed', i.e. fired in. For several hundred years, the images on some panes for stained glass windows have been painted on using enamels and fired in.

In terms of bulk quantities of glass for buildings, enamelling has developed since toughened glass was developed. The toughening process, where the glass is heated up to around 600°C and quickly cooled in a controlled way, is ideal for the relatively cheap and quick production of enamelled glass.

The earliest of these products, which became regularly available in the 1950s, and is still regularly produced, is simply painted all over with an emulsion of glass frit using any appropriate technique, such as curtain coating or spray painting. The emulsion is left to dry and the product is then passed through the toughening process. The frit, made from a glass with a lower melting point than the substrate glass, melts during the heating part of the toughening process and melds with the surface layers of the substrate. A frit which is based on lead glass is technically the best for this, but is being replaced by zinc glass since lead materials in powder form are recognised as being a serious health hazard. In the quench, the frit sets and becomes an integral part of the product. These enamelled glass products are regularly used in the spandrel areas of curtain walls, since they have sufficient opacity to conceal any building structure behind them.

A variant of the process, used since the 1970s, is screen printing (**Figure 10**), where a technique borrowed from the textile industry is used to lay down patterns of frit emulsion on the glass surface rather than cover the whole surface. The subsequent processing, drying and toughening, is the same for enamelled screen printed glass as it is for painted enamelled glass.

Figure 10 Screen printing (courtesy of Hansen Glass)

Solar control glass

It was known from the very earliest days of glassmaking that adding small quantities of certain materials caused the glass to have different colours. Colour has been used to produce decorative glass containers for thousands of years and stained glass windows for hundreds of years.

In the 1950s the increasing prevalence of large office blocks with a high proportion of glazing caused the buildings to overheat in direct sunshine. This was the case in many climates. To try and offset this, use was made of coloured body-tinted glass, mainly green, blue, bronze or grey, which became the first types of solar control glass. These products work by absorbing part of the solar energy (**Figure 11**). This makes the glass warmer and the warm glass then loses heat by radiation and convection. Since air movement is much greater on the exterior of the building, the majority of the energy absorbed by the glass is lost by convection to the exterior.

Using the same body tinted glass as the outer pane of an insulating glass unit (**Figure 12**) makes it considerably more effective, as the presence of an insulating cavity reduces the amount of absorbed energy which reaches the inside of the building.

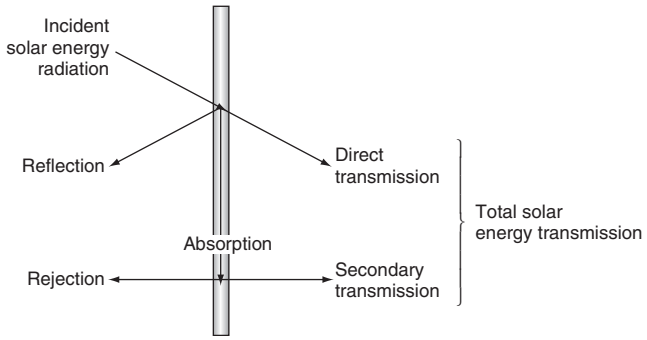

Figure 11 Solar energy diagram for single glass

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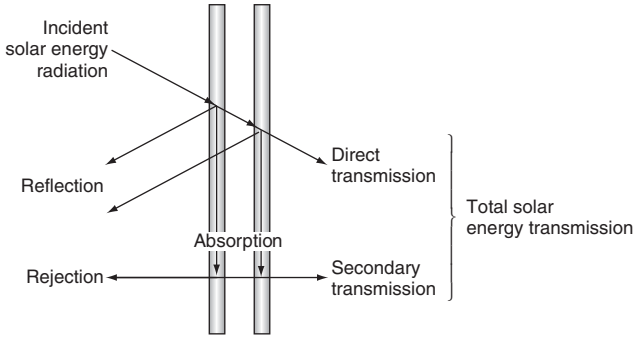

Figure 12 Solar energy diagram for double glazing

The nature of the tinted glass products means that they cannot be very efficient for solar control. They can be reasonably effective, simply by using more intense tinting (the simplest way being to use thicker glass), but this is at the expense of the light transmission.

Figures 11 and **12** give the main clue as to one way of making a better performance solar control glass – by increasing the amount of solar energy that is reflected. Since this energy is not absorbed by the glass it cannot contribute to the secondary transmission. The fast adoption of coated glass, which can have much higher reflection than body tinted glass, was a direct result of the need for better solar control glass.

The need for increasingly more effective solar control directed the early development of solar control coatings towards products with high reflection and low direct transmission. The early coatings had a similar effect on the visible part of the solar spectrum as they did on the total solar spectrum, leading to glass types which had high light reflection and low light transmission. Such was the need for effective solar control that some of the products had a light transmission less than 10% in order to achieve total solar energy transmission less than 20%. The appearance of buildings extensively glazed with these types of glass was like dark mirrors, giving rise to the derogatory term 'black buildings'.

The most efficient types of solar control glass are selective in their effect on the solar spectrum, i.e. they have much more effect on the infrared radiation than they do on the visible radiation. The development of 'high performance' coatings, based on silver layers in the coating, revolutionised the appearance of buildings with large areas of glass. Once these high-performance coated glass types became available, architects very quickly demanded glass types which would give good solar control performance without compromising the light transmission. This is not always feasible, since light is part of the solar energy, approximately 50% of it, so there is a limiting ratio of around 2:1 between the light transmission of a glass product and its total solar energy transmission.

In **Figure 13**, the shaded area gives the possible combinations of light transmission and total solar energy transmission for glass. Some of the 'high performance' glass products are close to the theoretical limit, maximising light transmission in relation to total solar energy transmission.

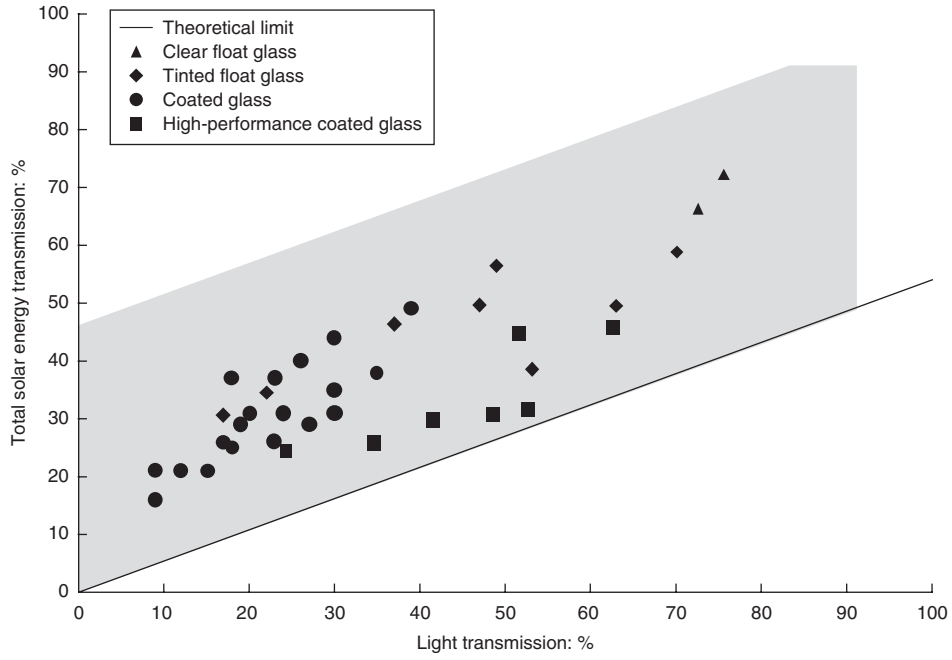

Figure 13 Light transmission and total solar energy transmission of various glass products

Coated glass

Definitions

The term coated glass could be used to describe any number of types of glass where a base glass has something applied to the surface. The term coated glass as currently used in the glass industry is limited to those types of glass where inorganic materials such as metals or metal oxides are applied to the glass surface. The term does not include enamelling, i.e. firing other glass types on to the surface of a base glass.

Coated glass can be divided into two major classes. Online coated glass is manufactured during the process of making the base glass. Offline coated glass is made by taking an existing piece of base glass and applying a coating to it.

Coated glass is covered by BS EN 1096, which classifies coatings into four types. Class A coated glass is sufficiently durable that it can be used on the exterior surface of buildings. Class B coated glass is slightly less durable but can be used exposed to the atmosphere inside buildings. Class C coated glass needs to be protected by incorporation into an insulating glass unit within a few days or weeks of being manufactured, or the coating will deteriorate. Class D coated glass is so vulnerable to atmospheric degradation that it needs to be permanently protected from the atmosphere within hours of being manufactured.

Vacuum deposition coating

This offline process was developed in the 1960s. The glass is placed on stillages in a vacuum chamber and coils of metal are placed around the glass. The vacuum chamber is taken down to an extremely low pressure and the coils of metal are heated by passing electricity through them. At the very low pressure, the metals evaporate and condense on the glass surface. By varying the metal and the coating thickness, various colours can be achieved.

The resulting glass product has a very thin metal coating on it, sufficiently thin to be transparent, but also with significant properties of reflecting or absorbing light and solar energy. The main purpose of these types of coated glass is solar control, which became a major issue in the 1950s and 1960s when office blocks with significant areas of glazing were found to easily overheat.

Vacuum deposition coated glass is Class D to BS EN 1096. Being very thin, any small amount of tarnishing due to exposure to a normal atmosphere causes the coating to change colour, so these coated glass types can only be used in products where they are protected from contact with a normal atmosphere. In most cases this has to be done within a few hours of production. One way of doing this is to incorporate the coated glass into an insulating glass unit immediately after it has been made, with the coating protected inside the dry atmosphere of the insulating glass unit. Another method is to immediately form the product into a laminated glass, with the coated surface in contact with the interlayer.

Vacuum deposit coatings have been superseded by more modern coatings.

Dip coating

One way of manufacturing coated glass offline is simply to suspend the glass and dip it into a tank of liquid which leaves a thin coating on the glass after it is removed. One product made in this way since the 1960s is a low reflection glass, called Mirogard or Amiran, made by Schott. In this process, both sides of the glass are coated simultaneously. The coating material is transparent, but has a lower refractive index than glass, so the amount of surface reflection, which depends on the refractive index, is very much lower than that from a normal glass surface. Its use is where it is important that the view through the glass is not disturbed by reflections, for example in display cases and in some shop windows.

The Amiran coating is relatively prone to physical damage and it does not conform to either Class A or Class B of BS EN 1096. It is given a special class, Class S, in BS EN 1096, since it has very specific, limited, applications.

Online coating

Online coatings were developed in the 1970s. By positioning sputtering bars at the end of the tin bath, where the glass is at a temperature of around 600°C, various materials can be deposited on the glass surface just before it enters the annealing lehr. The earliest coatings laid down in this way contained silica. By controlling the amount deposited, the coated glass could be given various levels of silver-coloured or straw-coloured reflection to produce effective solar control products. Because the coatings are deposited at high temperature they are hard and durable at normal temperatures and the glass products are generally Class A coatings to BS EN 1096. The coated glass can also be toughened, because the coatings are not adversely affected by being heated to 600°C.

In the early 1980s, Pilkington produced Pilkington K Glass, an online coated glass with low emissivity, by depositing tin oxide on the glass. Although the emissivity of this glass, at 0.16, was not as low as could be achieved with offline sputter coated glass (see next section), it became an instant success, because panes cut from stock sheets could be toughened by any competent processor. It was quickly copied by a number of other float glass manufacturers.

Recent developments in online coating have led to 'self-cleaning' glass and glass with a low-reflection coating. The latter only becomes a low-reflection glass when it is laminated with the low-reflection coating on both external surfaces.

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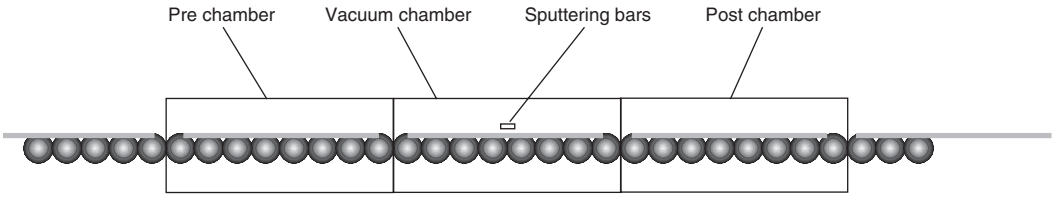

Figure 14 Sputtering coating process

Online coatings are sometimes also referred to as pyrolitic coatings (being produced at high temperature) and as 'hard' coatings.

Sputter coating

Most modern coated glass is made by a process developed by British Oxygen Corporation in the late 1970s. The process consists of a line of chambers through which the glass is passed on rollers (**Figure 14**).

The functional part of the process are the vacuum chambers containing the sputtering bars. The vacuum is low, but not as low as that required during the vacuum deposition process (see earlier). The process is continuous so when glass passes into a chamber a slot opens for the glass to pass through, which closes afterwards. Air enters the chamber as this happens, so there are a number of chambers, before and after the sputtering chambers, which are there to limit the amount of air lost and to keep the pressure in the sputtering chambers as steady as possible. In the sputtering chambers, a plasma of the sputtered material is formed and this condenses on the glass surface as it passes through. If several sputtering chambers are installed, coatings can be applied in several layers.

The first coated glass types produced using this process had only one coating material layer applied, which was a relatively hard metal oxide, metal nitride or durable metal. The natural durability of these materials makes them Class B to BS EN 1096. They were designed to provide solar control by means of increased solar energy reflection. Typical colours were blue, silver and bronze.

The colour developed by the coating is not so much due to the colour of the coating material, but by selective optical interference due to the thickness of the various layers in the coating. An analogy to this is found in the appearance of thin layers of (transparent) oil on (transparent) water, which can give a variety of colours in reflection, depending on the thickness of the oil layer.

Extension of the sputtering process plant to more chambers allowed generation of a superior vacuum in which it is possible to deposit more sensitive materials. In around 1980, coated glass with a silver metal layer was made available, the silver being sufficiently thin that it was relatively transparent. This was the first low-emissivity (low E) glass (see section on Thermal insulation earlier in this chapter) and led to a major change in the performance of insulating glass units used in buildings. The use of silver for this purpose, despite the fact that it tarnishes relatively easily on exposure to air and has relatively poor adhesion to the glass substrate, is because it has a surface emissivity lower than any other material. The silver-based low E glasses gave rise to the term 'soft coated'. They can only be used in insulating glass units because they tarnish and abrade so easily (Class C to BS EN 1096). Because the coating adhesion is so poor, it has to be removed where the adhesive part of the insulating glass unit edge seal contacts the glass (a procedure known as edge-stripping or edge-deletion).

Further development of the sputtering process has led to higher numbers of chambers being used. (The longest line currently in production has 87 chambers.) The larger number of sputtering chambers, and the better coating control that these longer production lines provide, have led to coatings of considerably more sophisticated layerings being used, with consequent improvements in the performance of the glass. The use of two (or more) separate layers of silver in the coating, instead of one, not only improves (lowers) the emissivity while giving better optical transparency, but can also be used to selectively reflect the infrared portion of the solar energy spectrum, giving rise to a range of products, generally known as 'high performance' coated glass, which have a relatively high visible light transmission, a very low emissivity, and solar control which is close to the optimum.

One disadvantage of sputter-coated glass is that the coatings are generally not able to survive in the toughening process. At 600°C, they scorch. The only way to get such glass products in toughened form is to apply the coating to a pane of glass which is already toughened. Since the glass has to be cut to the correct size before toughening (and the correct size is often not known until relatively late in a contract), this inevitably results in long lead times. Since the late 1990s, developments in sputter coating have given rise to a number of products, both low E and high performance, that are provided with a protective top coating to stop the functional part of the coating deteriorating during toughening. This potentially reduces lead times, since stock sheets of the coated glass can be cut to size and toughened. The protective top coat has no effect

on the resistance of the functional coatings to abrasion or tarnishing. These types of toughenable coated glass, with a very low emissivity, are now preferred to online coated low E glass since they give better thermal insulation.

Sputter coatings are also known in general as 'soft' coatings (wrongly, since some of the coatings are relatively hard).

Coated laminated glass

Most coated glass can be obtained in laminated form, but there are some limitations which need to be borne in mind.

Coated glass which meets Class A and Class B of BS EN 1096 is sufficiently robust to be processed by any of the laminating processes. Coatings which are Class C to BS EN 1096 are generally too easily abraded to go through any of the laminating processes. Coated laminated glass with Class C coatings has to be made by applying the coating to laminated glass. The interlayers of laminated glass are plastic and contain materials which will gas off in the vacuum chambers, potentially disrupting the plasma field. Most coated glass manufacturers will restrict the interlayer to no more than 0.76 mm thick and may also refuse to coat laminated glass with interlayers other than pvb if they are not sure of the quantity and type of material which may gas off.

Class A and Class B coatings which are purely solar control may possibly be positioned against the interlayer as they may still function well in that position. However, the coatings may not adhere to the interlayer material as well as glass does and caution should be used when considering coated laminated glass in this form.

Low emissivity is a surface effect. If the surface is covered up then it has no effect. The low emissivity properties of low E glass are specifically targeted at long wave radiation from objects at around room temperature, to which glass is completely opaque. The low-emissivity coating must be an exposed surface of the laminated glass to function correctly.

Fire-resistant glass

Wired glass

The earliest fire-resistant glass was wired glass (see section on Wired glass earlier in this chapter). So successful has this product been that it is still holds a major share of the market for fire-resistant glass.

Borosilicate glass

In the 1950s, borosilicate glass (which has borax, B_2O_3 instead of lime, CaO, and is closely related to glass ovenware in composition) became available as a building product. It has a higher softening point that soda lime silicate glass and its naturally very low coefficient of thermal expansion means that, appropriately framed, it does not crack in fires and it makes a good fire-resistant glass. Despite being aesthetically more pleasing, since it has no wires, and also being capable of giving longer fire resistance because of its higher softening temperature, the cost differential means that traditional wired glass still has a larger share of the market.

Borosilicate glass is covered by BS EN 1748-1.

Sodium silicate interlayers in laminated glass

In the early 1980s, BSN (a European glass company which incorporated what is now Glaverbel, part of what is now St Gobain Glass, and what is now Pilkington Germany) developed an interlayer which would intumesce (turn into a foam) when exposed to fire conditions. The interlayer was basically sodium silicate with about 15% water content. When heated, the water simply boils. The resulting glass not only holds together when exposed to fire conditions (integrity) but also is effective in stopping the passage of radiant heat (insulation). This gives it a big advantage over existing integrity-only fire-resistant glass products, such as wired glass and borosilicate glass, as these products remain clear in a fire situation, allowing large quantities of radiant heat to pass through.

Products based on this technology allow the unrestricted use of glass in 'compartment walls' where the it had previously either been excluded or allowed very limited in area.

Other fire-resistant glass

Borosilicate glass gets its fire resistance because of its low coefficient of thermal expansion and higher softening temperature. Other glass compositions can also have low coefficients of thermal expansion and higher softening temperatures.

Alkaline earth silicate glass (BS EN 14178) has no sodium in the composition but has additions of strontium, barium, potassium, aluminium and zirconium oxides.

Glass ceramics (BS EN 1748-2) have only very small proportions of sodium and potassium oxides, but a mixture of many other oxides is added to the silicate glass. It is different from all the other types of glass discussed in this chapter as it is partially devitrified. The resulting product has a thermal expansion coefficient which is very close to zero over a wide temperature range. In conventional fire tests it is unaffected. Glass ceramics products are more commonly seen as the glass tops of cooker hobs.

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Chapter 69

Strength and durability of glass

Chris Jofeh Arup, UK

Glass is a surprisingly strong isotropic homogeneous material that behaves in a perfectly linear elastic manner until it breaks or buckles. It does not yield, and fails without warning. It has good durability against most chemicals, but surface imperfections significantly reduce the stresses that the glass can safely carry. Heat treatments have been developed that introduce permanent compressive stresses into the surface of sheets of glass, thus inhibiting crack growth and glass failure. Because glass is a brittle material, it is important that glass structures are configured to 'fail safely'. This means that the breakage of a single piece of glass should not precipitate a sudden collapse, or allow that piece of glass to fall without warning onto people below. Glass design techniques aim to control the likelihood of breakage to an acceptable level and mitigate the consequences. Testing will often play an important part in the development of a structural glass design.

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Introduction

The purpose of this chapter is to describe for designers issues which codes or standards for other structural materials deal with, but which are yet to be thoroughly codified in glass: approaches to designing with a brittle material; safety; testing; and laminated behaviour. Its aim is to help designers understand what is different about the material, so that they can design appropriately. In many cases, codes will not be available for designers, who must therefore design from first principles. An understanding of how glass differs from more familiar materials is an essential starting place.

Physical properties of glass, compared with other materials

Glass is an isotropic homogeneous material that behaves in a perfectly linear elastic manner until it breaks or buckles. **Table 1** compares some of the properties of soda-lime silica glass with other, more familiar, structural materials. The Poisson's ratio of soda-lime silica glass is about 0.23 (Haldimann *et al.*, 2008). Glass is a brittle material (Creyke *et al.*, 1982) and does not yield: when tensile stresses exceed a value that is determined by the condition of the surface of the glass, it breaks suddenly and without warning.

Figure 1 shows the results of testing of 741 samples of float glass measured by CEN/TC129/WG8 to establish the characteristic strength of float glass. The samples were $1000 \times 1000 \times 6$ mm float glass and were loaded by a coaxial double ring. The graph clearly shows the wide variability in strength that can be found in the same material, and how much lower it is than the theoretical strength of glass. This is caused by variations in the surface condition of the glass.

Durability

The durability of glass is affected by its environment. Glass has good chemical resistance against water, saline solutions, acids, alkalis and organic solvents, and in this respect surpasses the majority of plastics. It is attacked by hydrofluoric acid and, at elevated temperatures, by concentrated phosphoric acid. Alkalis attack glass surfaces as concentration and temperature increase. Glass with alkali attack exhibits pinholes and a matt surface finish that resembles fine sandpaper.

The surface of a piece of glass will contain cracks. These may be too small for the eye to see, and can be caused by abrasion from wind-blown dust or by scratching, and even by vigorous cleaning. Any mechanical agent (for example vibration or imposed deformation or impact) or uneven heating can cause tensile stresses which in turn can cause cracks to propagate, which can eventually break the glass. Irreparable glass damage can occur from improper storage and handling, to which glass edges are especially vulnerable, and cracks often propagate from damaged edges.

Glass can fail spontaneously because of the action of minute inclusions of nickel sulphide, as described in Chapter 5. *Glass in Buildings: Breakage – the Influence of Nickel Sulfide* (CWCT, 2002) considers the use of toughened glass in buildings and the effects of nickel sulphide on the risk of glass failure. It covers the risk of failure of toughened glass, the process of heat soaking and the appropriate use of glass.

Glass will pit and damage badly if weld spatter containing molten metal particles lands on the glass. Welding on metal

Material	Young's modulus: kN mm^{-2}	Strength†: Nmm^{-2}	Coefficient of thermal expansion: K^{-1}	Density: kgm^{-3}	Embodied energy: MJkg^{-1}	Fracture toughness: MPam$^{0.5}$
Glass	70	5000*	8–9 × 10^{-6}§	2500	13–23	0.8
Carbon steel	205	200–2000	12 × 10^{-6}	7850	50–60	20–200
Aluminium	70	40–600	25 × 10^{-6}	2800	290–305	15–50
Concrete	20–40	10–100	12 × 10^{-6}	2400	3–6 (8–20 when reinforced)	0.2–0.3

Notes: † Strength: metals – yield strength; glass – tensile strength; concrete – compressive strength

* See Chapter 67 of this section for further information about the strength of glass, and how the surface condition of the glass significantly reduces the in-service strength of the glass

§ Borosilicate glass has a lower coefficient of thermal expansion, from 3–6 × 10^{-6}

Table 1 Typical material properties of soda-lime silica glass compared with other structural materials (adapted from Ashby, 2005)

near glass can cause glass to shatter because of the significant rate of temperature change.

The polyvinyl butyral (PVB) interlayers commonly used in laminated glass have a tendency to wick moisture, discolour and not age well.

Variations in atmospheric pressure, air temperature and solar gain induce changing stresses in the edge seals of insulating glass units (IGUs), commonly known as double-glazing, and the materials of the seals themselves are in general subject to long-term degradation as a result of attack by paints, cleaning fluids and atmospheric pollutants (Garvin and Blois-Brooke, 1995). In service, moisture will naturally permeate through the edge seals of a double-glazing unit, eventually causing the dew point of the air within the unit to rise and leading to failure by misting (the condensation of water on the inner face of the outer pane when the outside temperature is low). IGUs have been known to perform satisfactorily for up to 35 years (Loughran, 2003).

Design approaches

Because glass is a brittle material, it is important that glass structures are configured to 'fail safely'. This means that the breakage of a single piece of glass should not precipitate a sudden collapse, or allow that piece of glass to fall without warning onto people below.

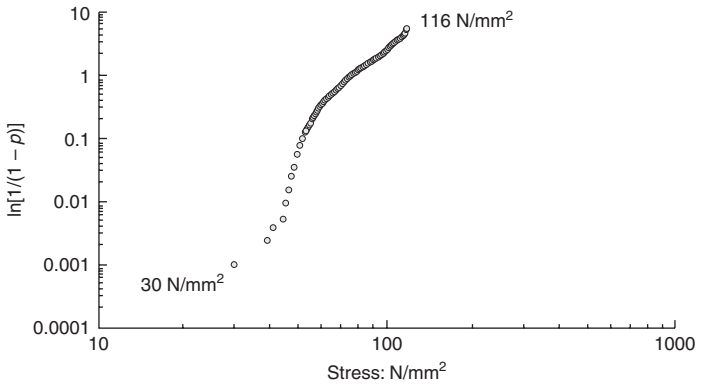

Figure 1 Weibull plot of the failure stresses of 741 samples of float glass

Redundancy is an approach often used in glass design. This provides more than one load path, so that loads can still be carried if a piece of glass breaks. Redundancy may be in the configuration of the structure or in the use of laminated glass where each ply is capable of carrying the design loads, or both.

Robustness is a desirable characteristic of any structure, and is generally considered (Starossek and Haberland, 2008) to be the avoidance of disproportionate damage to an initial disturbance. With glass, it may be achieved by a combination of protection (of vulnerable edges, for example), care with details and redundancy. Over-design will rarely be sufficient. A task group of the Institution of Structural Engineers is preparing a *Working Engineers' Guide on Robustness and Disproportionate Collapse*, and it is anticipated that this will be published in late 2009.

Surface imperfections significantly reduce the stresses that annealed glass can safely carry. Heat treatments have been developed that introduce permanent compressive stresses into the surface of sheets of glass, thus inhibiting crack growth and glass failure. The two main types of heat-treated glass are known as heat-strengthened glass and toughened or tempered glass. These are described in more detail in Chapters 67 and 68 of this section. Typical surface compressive stresses will be around 40 Nmm^{-2} for heat-strengthened glass and 100 Nmm^{-2} for toughened glass. Standards vary around the world, so it is important to agree with your manufacturer what surface compressive stresses will be achieved.

Care with details is an important aspect of glass design and manufacture. Because glass does not yield, but breaks without warning under tensile stress, the strength of connections, which may contain stress raisers, must be established carefully, and the structure configured so that the loads carried by the connections can be predicted accurately. At Parc de La Villette in Paris, the designers of the glass walls developed bespoke ball-and-socket bolted

connections that eliminated unpredictable local bending stresses in the glass (Rice and Dutton, 1995).

Testing will often play an important part in the development of a structural glass design, and there can be many reasons why testing is needed. The decision to test, and the nature and extent of testing, will be influenced by consideration of:

- cost
- whether there are precedents
- consequences of failure
- degree of redundancy
- extent and nature of warning of distress in glass
- anticipated stress levels
- relevant experience of designer
- relevant experience of manufacturer and erector/installer
- quality assurance systems of manufacturer and erector/installer.

The checking authority or insurer or designer may require testing:

- to assess likelihood of failure
- to assess consequences of failure
- because calculations may be unfeasible.

Development tests (see **Figure 2**) may be required:

- to assess what range of strengths and resistances a detail has
- to check the appearance of a detail
- to check the buildability of a detail
- to compare different details
- to identify critical areas.

Tests on glued joints may be required:

- to check the quality of an adhesive
- to determine the effectiveness of surface preparation
- to gather data for the prediction of joint behaviour
- to select an adhesive from a group of candidates
- to evaluate the effect of ageing.

Proof tests may be required:

- to weed out weak components
- to provide a basis for probabilistic design
- to confirm adequacy of calculations
- to determine post-fracture behaviour.

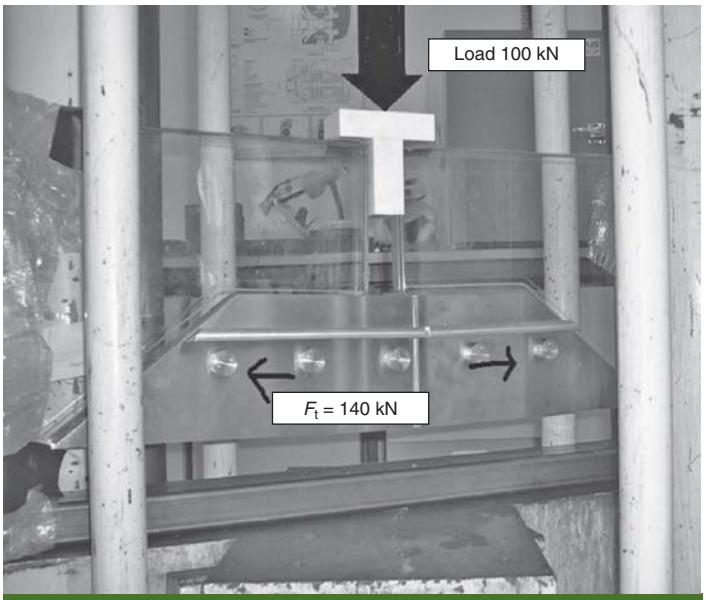

Figure 2 Development test on glass staircase connection detail (courtesy of Arup)

Production tests may be required to check the quality of production (reflecting the critical areas identified during development testing).

Safety

Glass in any form can break in service from a number of causes including impact, poor workmanship, thermal stress and entrained particles. As in any other field of engineering, absolute safety, meaning zero risk, cannot be achieved. Glass design techniques aim to control the likelihood of breakage to an acceptable level and mitigate the consequences.

Glass breaks in different ways, according to its degree of heat treatment. Typical breakage patterns are shown in Figure 1.10 of Haldimann *et al*. (2008). The way that the glass breaks influences the detailing required to ensure that the glass does not fall from its supports as soon as it is broken.

Annealed glass breaks into jagged shards with sharp edges if impacted or overloaded. Heat-strengthened glass, when produced by a properly controlled process, breaks in a similar way to annealed glass, although cracks always run out to the edges. Toughened (fully tempered) glass breaks into small roughly cubic fragments ('dice') whether impacted, locally damaged by a sharp object or broken spontaneously by an inclusion. Both toughened and heat-strengthened glasses are more resistant than annealed glass to thermal stresses, thanks to their surface compressive stresses. The manufacture of these products is described in Chapter 68 of this section, and their properties are described in Chapter 70.

Falling glass

It is widely recognised that glass falling from buildings is hazardous. The level of the hazard depends principally on the likelihood of people being in the area where it may fall, and the height from which the glass can fall.

Vertical glass can be at risk of falling after breakage if the broken pieces do not remain in place, either because its attachments no longer work or because of external forces such as gusts of wind.

The probability of glass falling is naturally higher when it is inclined from the vertical and its own weight contributes to forces that would displace it.

Overhead glazing has been defined in the UK for some time as glass inclined at more than 15° from the vertical. There is no recorded reason for the selection of 15°, and other countries have different definitions of overhead or sloped glazing.

It has been common past practice in the UK to use monolithic toughened glass overhead on the basis of the following arguments:

- Toughened glass is much stronger than normal annealed glass and should therefore be more resistant to breakage.
- Toughened glass is classified as 'safety glass' under BS 6206: 1981, meaning that when impacted by a human, it breaks comparatively safely for the person hitting it, compared with annealed glass.
- The risk of toughened glass breaking spontaneously can be controlled by good glazing practice and also heat-soak testing to reduce the presence of damaging inclusions.

The validity of this reasoning has been questioned recently because:

- Toughened glass can fall in large clumps of particles that can cause serious injury.
- Even if not 'clumped', the falling fragments can cause serious injury by knocking people to the ground. Having 200 kg of glass fragments fall two stories onto your head is no safer than having 200 kg of gravel do the same thing.
- There remain many causes of breakage and some methods of heat-soak testing have not proved as reliable as expected.
- Individual buildings may contain sufficient quantity of toughened glass to present an unacceptably high probability of breakage despite the highest levels of reliability for each pane.
- Falling glass has a very powerful impact on people's perception of safety and, when confidence is lost, the actual rate of failure and risk of injury become academic.
- The operational and economic costs of loss of confidence in a building following glass breakage have been found recently to be significant.

A falling piece of laminated glass could potentially do greater harm than falling toughened glass, so the reliability of its retention needs to be carefully designed for service conditions. Laminated glass, whether of annealed, heat-strengthened or toughened plies, should be designed and framed securely to ensure that, if broken, it will remain in place.

Roof glazing in the UK must comply with *Test for Non Fragility of Roofing Assemblies* (Advisory Committee for Roof Work, 2005), also known as the Red Book. It requires that roof glazing must be non-fragile, which means that if someone falls onto the glazing in the course of normal access or maintenance, he or she should not fall through it.

Safety and Fragility of Glazed Roofing – Guidance on Specification and Testing (CWCT, 2004) gives guidance on glass roof design to avoid fragile construction, and sets out an outline testing regime.

Designers of glazed roofing may additionally choose to apply criteria for robustness against hard body impacts to represent scenarios such as dropped tools or equipment according to the situation.

Designers and manufacturers of bespoke structural glass assemblies need to pay particular attention to the connection details of glass components and the prevention of progressive collapse. Structural glass connections are not standardised or codified and their designs must be individually developed and tested, allowing for achievable control of manufacturing variables.

If novel combinations of glass types, heat treatment conditions, coatings or laminating interlayer materials are to be used then the interaction of materials and manufacturing processes must be studied and reflected in the design, testing, specification and quality control.

The final 'watch-it' about glass design is the need for designers to consider replacement. Whereas installation of glass during construction may be a relatively straightforward affair, removal and replacement in service may be much harder. If glued connections are to be used, the designers must consider and record how broken glass can safely be removed, the surfaces primed and the new glass adhered.

Rules of thumb

Engineers sometimes find it helpful to use rules of thumb in preparing preliminary designs and in checking designs. **Table 2** lists the characteristic bending strength and permissible tensile stress for a range of glasses (Wurm, 2007). These values should be used with caution, and designers should never forget that it is peak stresses that cause glass to break, so analytical or testing methods must be used which predict or induce peak stresses. prEN 13474-1 gives draft guidance on the calculation of effective and allowable stresses in glass sheets subjected to out-of-plane bending.

The most commonly available form of glass is in thin sheets, so considerations of stiffness (stability, deflection,

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Glass type	Characteristic bending strength: Nmm^{-2}	Permissible tensile stress: Nmm^{-2}
Tempered (toughened) float glass	120	50
Tempered rolled glass	90	37
Annealed glass	45	12
Heat-strengthened float glass	70	29

Table 2 Characteristic and permissible strengths of different glasses (data from Wurm, 2007). These data are based on a deterministic approach with a global safety factor and the use of unfactored loads

vibration, buckling) may govern over strength. The glass fins that support many glass walls are a good example of this, as illustrated in **Figure 3**.

IStructE (1999) gives a simple method of assessing the buckling of the unrestrained edges of glass fins, based on Bleich (1952). Haldimann *et al.* (2008) point out that, as it takes no account of imperfections or laminated behaviour, it should only be used for preliminary design. The simplified approach says that the destabilising moment of a monolithic glass fin of thickness t is given by

$$M_{max} = Et^3/[6(1+\nu)]$$

which includes a factor of safety of 2.

Tests by Pilkington have shown that this value of M_{max} is valid for suspended simply-supported and cantilever fins, restrained in torsion at their end fittings and glazed using bolted fittings with lateral movement restricted to 5 mm.

Laminated glass

The manufacture of laminated glass is described in Chapter 68 of this section. When glass is laminated, the interlayer helps the individual sheets of glass to work together. This may be monolithically, or it may be as layers that deflect together, as shown in **Figure 4.**

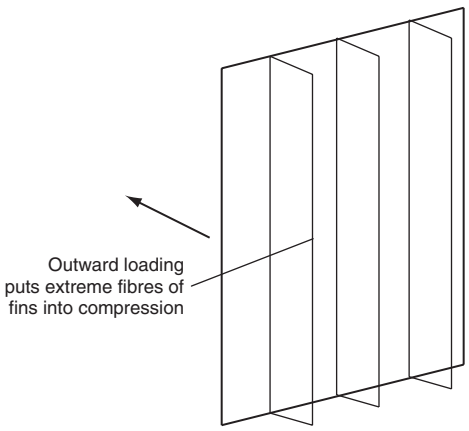

Figure 3 Destabilising moment applied to simply-supported glass fins

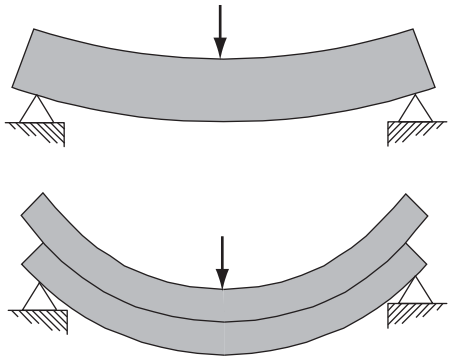

Figure 4 Monolithic and layered behaviours

In monolithic behaviour, the stiffness of the pane is based on the total glass thickness, whereas in layered behaviour, the total stiffness is simply the sum of the individual stiffnesses.

Consider the triple laminate shown in **Figure 5**.

Monolithic behaviour $I = b(t_1 + t_2 + t_3)^3/12$

Layered behaviour $I = b(t_1^3 + t_2^3 + t_3^3)/12$

The behaviour of laminated glass is codified by the draft European standard prEN 13474-1, which introduces the shear transfer coefficient of the interlayer. The shear transfer coefficient will vary with polymer chemistry and is influenced by temperature, rate and duration of loading. Designers wanting to model the behaviour of the interlayer will need to seek information about the different interlayer products from their manufacturers. Without such data, it may be appropriate to treat the glass as monolithic under very short-term loading and/or low temperatures, and as layered under all other load durations and warmer temperatures.

Polyvinyl butyral (PVB) is the commonest interlayer material, and is available in sheets, known as foils, that are 0.38 mm thick. Two, four or even six foils are used to form a single interlayer, depending on the unevenness of the glass panels that are being joined. Heat-treated glasses (i.e. toughened (tempered) or heat-strengthened) are generally more uneven than annealed glass. All will have varying

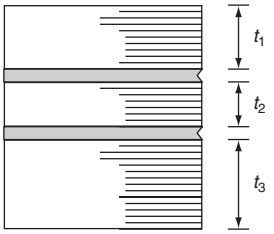

Figure 5 Laminated glass

Density	1070 kgm^{-3}
Shear modulus	0–4 GPa
Poisson's ratio	~0.5
Coefficient of thermal expansion	80×10^{-6} K^{-1}
Tensile strength	⩾20 Nmm^{-2}
Elongation at failure	⩾300%

Table 3 Typical material properties of PVB (Haldimann *et al*., 2008)

degrees of bow, roller wave and edge dip (IStructE, 1999). **Table 3** gives typical properties of PVB. Other materials that are used include PVB combined with polyethylene terephthalate, ethylene vinyl acetate foil, polyurethane foil, poured resin, polyester sheet, ionomers and more. Their properties are described in Chapter 68 of this section. **Figure 6** shows the triple laminated glass beams used at the *ss Great Britain* in its dry dock in Bristol. Three 10 mm sheets of heat strengthened glass, laminated using 0.76 mm pvb, are used in each beam.

The buckling of laminated glass

The buckling of laminated glass is influenced by glass thicknesses, initial deformations, load duration, the interlayer material and the boundary conditions (Haldimann *et al*., 2008). These authors analyse column buckling, lateral torsional buckling, and plate buckling.

Blaauwendraad (2008) discusses the buckling of laminated glass columns, and introduces a dimensionless parameter which models the effectiveness of the interlayer.

Figure 6 An example of redundancy – the triple laminated heat-strengthened glass beams supporting the 'Glass Waterline Plate' at the *ss Great Britain* in its dry dock in Bristol (courtesy of Arup/Lance McNulty)

Insulating glass units

Chapter 70 of this section describes aspects of the behaviour of insulating glass units, and their durability has been described earlier in this chapter. Appendix D of IStructE (1999) demonstrates how glass deflection causes volumetric change, leading to pressure change of the gas within an IGU, which helps to share the loads between opposite panes. It notes that some national standards allow only partial load-sharing, and that the stiffer the individual panes, and the deeper the cavity, the less the load sharing that takes place. An extreme instance of this is in curved or folded IGUs, which are sometimes used at the corners of buildings. In these circumstances, the curvature of the glass may make it so stiff that it attracts forces that the IGU is not strong enough to withstand, leading to premature failure.

IGUs are also subject to isochoric pressures, which are those pressures that result from changes in temperature or atmospheric pressure from the conditions that existed when the units were manufactured. Appendix E of IStructE (1999) provides further information about this phenomenon.

Concluding remarks

Glass is in many ways like durable transparent unreinforced high-strength concrete. It is usually supplied in thin sheets, though tubes are manufactured for the chemical industry that may be used structurally. Its brittleness means that particular care must be taken in designing and detailing glass structures so that they are adequately robust. Glass design techniques aim to control the likelihood of breakage to an acceptable level and mitigate the consequences. This is sometimes known as designing glass to 'fail safely'. Testing will often play an important part in the development of a structural glass design.

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Chapter 70

Common glass products

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At first glance there appears to be a myriad of glass products. In reality there are relatively few, each of which has variants. Most of these are covered by European Standards.

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Basic glass to EN 572

Float glass to EN 572-2

Float glass forms the bulk of glass used in buildings. It can be used in its basic form or it may be subjected to further processing. The float plants are commonly designed to produce standard size 'jumbo' sheets of glass, 6000 mm × 3210 mm.

For use in buildings, clear float glass is manufactured in the thicknesses shown in **Table 1**, together with the thickness tolerances.

Float glass is also manufactured in a range of body-tinted colours – bronze, grey, green and blue. Not all the thicknesses shown in **Table 1** are available for these body-tinted solar control float glass products, although all of them are available in 6 mm and 10 mm thicknesses.

Float glass is also manufactured with a very low proportion of iron, which removes almost all of the green tinge typically associated with clear float glass. This is generically called low iron glass, sometimes referred to as white glass.

Float glass has practically no local distortion, so vision through is normally undistorted. However, it is a mistake to consider it as being perfectly flat. The thickness can vary within the tolerance range across the width of a jumbo sheet. This thickness variation is unimportant for most glazing applications.

Polished wired glass to EN 572-3

Polished wired glass is made by grinding and polishing both surfaces of wired patterned glass. For use in buildings, polished wired glass is manufactured in the thicknesses shown in **Table 4**, together with the thickness tolerances.

Polished wired glass cannot be toughened.

Sheet glass to EN 572-4

Sheet glass has been largely superseded by float glass, but it is still manufactured. Its applications tend to be in areas where vision through and aesthetics are unimportant, such as commercial greenhouses.

Patterned glass to EN 572-5

Patterned glass is produced by the rolled glass process. For use in buildings, patterned glass is manufactured in the thicknesses shown in **Table 2**, together with the thickness tolerances. The glass manufacturers should be consulted about the pattern designs, sizes and thicknesses available.

Most patterned glass designs can be toughened or laminated, but the manufacturer should be consulted.

Wired patterned glass to EN 572-6

Wired patterned glass, also known as cast wired glass, is made by the rolled glass process. For use in buildings, wired patterned glass is manufactured in the thicknesses shown in **Table 3**, together with the thickness tolerances.

Wired patterned glass cannot be toughened.

Annealed glass

The term annealed glass covers glass products which are cooled at a sufficiently low rate that there is very little or no residual stress in the glass. The basic glass types covered by EN 572 are all types of annealed glass. Annealed glass is characterised by the ability to cut it by scoring the surface with a diamond or a cutting wheel and then snapping the glass along the score line. If glass is poorly annealed, cutting glass in this manner may be difficult.

Annealed glass is what we know as ordinary glass. It has a highly variable strength and when broken cracks into

Nominal thickness: mm	Tolerances: mm
2	±0.2
3	±0.2
4	±0.2
5	±0.2
6	±0.2
8	±0.3
10	±0.3
12	±0.3
15	±0.5
19	±1.0
25	±1.0

Table 1 Float glass thickness

relatively large pieces which may have lethally sharp edges. The lack of any internal macrostructure means the edges can be sharper than can be obtained with any metal.

The strength of annealed glass can be sufficiently low, particularly at the edges, that it becomes vulnerable to thermal stress cracking simply by being exposed to the sun in normal use (see section on *Annealed glass* in Chapter 71).

Toughened glass to BS EN 12150

Toughened glass to BS EN 12150 has two specific characteristics. It has to be manufactured with a surface compressive stress sufficiently high that it fragments into small dice when broken (and thus can be classified as a safety glass) and so that it has a reliable minimum bending strength. To get these two characteristics to meet the requirements of BS EN 12150, the average surface compressive stress needs to be at or above 80 N/mm^2. The quality assurance (QA) procedures laid down in BS EN 12150 are specifically designed to ensure that the product meets these requirements.

In addition to having an appropriate prestress built into it, BS EN 12150 requires that the glass has limited distortion, in terms of overall bow and local bow. The requirements in this respect are not onerous and most of the better manufacturers will meet them easily and may even claim tighter standards in their own literature. At the time of writing, there is no specific limit on roller wave distortion in toughened glass, but this is being considered in the next revision of the standard.

Nominal thicknesses: mm	Tolerances: mm
3	±0.5
4	±0.5
5	±0.5
6	±0.5
8	±0.8
10	±1.0

Table 2 Patterned glass thickness

Nominal thickness: mm	Tolerances: mm
6	±0.5
7	±0.5
8	±0.8
9	+1.5 – 1.0

Table 3 Wired patterned glass thickness

Toughened glass is available in thicknesses from 3 mm to 25 mm in float glass and 4 mm to 10 mm in patterned glass. All types of tinted float glass and patterned glass can be toughened. Some coated glass types can be toughened with the coating already on the glass, while other coatings have to be deposited on glass which is already toughened, but, either way, most types of coated glass can be obtained in toughened form.

Wired glass cannot be toughened since the wires cause it to disintegrate when the prestress is being developed.

The sizes of toughened glass available depend on the size of the manufacturing plant. Panes up to 2400 mm × 4500 mm are readily available. A few processors can manufacture up to 3000 mm × 6000 mm. Glass 6 mm and thinner may be restricted in size for reasons of handling.

Heat-strengthened glass to BS EN 1863

Heat-strengthened glass to BS EN 1863 also has two specific characteristics. It has to be manufactured with a surface compressive stress sufficiently high that it has a reliable minimum bending strength and sufficiently low that it does not fragment into many pieces, i.e. so that it breaks like annealed glass (see section on *Annealed glass* in this chapter). To get these two characteristics to meet the requirements of BS EN 1863, the average surface compressive stress needs to be between 30 N/mm^2 and 55 N/mm^2, depending on thickness (the thicker the glass, the lower the surface compressive stress needs to be to

Nominal thickness: mm	Limiting values: mm	
	Minimum	Maximum
6	6.0	7.4
10	9.1	10.9

Table 4 Polished wired glass thickness

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prevent fragmentation). The QA procedures laid down in BS EN 1863 are specifically designed to ensure that the product meets these requirements.

In addition to having an appropriate prestress built into it, BS EN 1863 requires that the glass has limited distortion, in terms of overall bow and local bow. The requirements in this respect are not onerous and most of the better manufacturers will meet them easily and may even claim tighter standards in their own literature. At the time of writing, there is no specific limit on roller wave distortion in toughened glass, but this is being considered in the next revision of the standard.

Heat-strengthened glass to BS EN 1863 is generally available in thicknesses from 3 mm to 10 mm in float glass. Thicker glass is extremely difficult to produce within the strength and fragmentation limits of BS EN 1863. The US standard for heat-strengthened glass (ASTM C1048-04 Standard Specification for Heat-treated Flat Glass – Kind HS, Kind FT Coated and Uncoated Glass) does not limit fragmentation of heat strengthened glass, so it allows thicker glass which can be considered as heat strengthened glass. ASTM C1048–04 is not applicable in Europe. When considering heat strengthened glass, care must be taken to ensure the appropriate standard is referred to.

The sizes of heat strengthened glass available depend on the size of the manufacturing plant. Panes up to 2400 mm × 4500 mm are readily available. A few processors can manufacture up to 3000 mm × 6000 mm. Glass 6 mm and thinner may be restricted in size for reasons of handling.

Heat-soaked toughened glass to BS EN 14179

The strength, breakage characteristics and distortion limits are the same for heat-soaked toughened glass to BS EN 14179 as for toughened glass to BS EN 12150. The difference between the two is a second process (heat soaking) which is designed to eliminate the vast majority of those panes of toughened glass which may otherwise fail in service from nickel sulphide inclusions. Because of the way nickel sulphide reacts to heat, the requirements for heat soaking the glass are explicitly stated in BS EN 14179 and the product can only meet the standard if it is heat soaked in an oven in such a way that the whole area of every pane of glass in the oven achieves a temperature between 280°C and 300°C (not higher or lower). This is extremely onerous for oven design when the heated space has to be capable of accommodating panes up to 6 m × 3 m, so BS EN 14179-2, which deals with means of ensuring the product conforms to the standard, has a set of detailed tests which the heat-soaking process must meet to demonstrate its effectiveness.

Heat-soaked toughened glass is available in the same glass thicknesses, types and sizes as ordinary toughened glass.

Enamelled glass

There are no standards specifically for enamelled (painted or screen printed) glass. The products are either toughened glass (see earlier sections in this chapter), heat-strengthened glass (see earlier sections in this chapter), or heat soaked toughened glass (see earlier sections in this chapter).

Not all thicknesses may be available. The heating time for glass less than 6 mm thick is insufficient to completely melt some types of enamel, so enamelled glass (either painted or screen printed) is not commonly available less than 6 mm thick. Painted glass, with its thick enamel, may cause problems if the enamel is kept soft for too long, so painted enamelled glass more than 12 mm thick is unusual due to the length of time needed to heat thicker glass.

Laminated glass to BS EN ISO 12543 and BS EN 14449

General

Laminated glass is available in many different forms and a large range of thicknesses for different applications. Most monolithic glass types discussed in sections on basic glass, toughened glass, heat-strengthened glass, heat-soaked toughened glass and enamelled glass can be incorporated into laminated glass, as also can polycarbonate sheets. Laminated glass is used for several different purposes.

Safety

The original purpose of laminating was safety. Simply by using an interlayer material with good energy absorption and a high extension to failure, the glass tends to hold together after fracture, meaning that the lethally sharp cutting edges of broken glass are not exposed. The glass only performs in this way if the glass is not penetrated and stays in its fixing system, so its performance as a safety glass depends on the way it is fixed as well as the properties of the material.

Security

Given a sufficient number of glass plies and thickness of the energy-absorbing interlayers, laminated glass can be very difficult to penetrate. Even a three-ply laminated glass incorporating a single interlayer of sufficient thickness can give a reasonable level of security against a ‘smash and grab’ attack. For resistance to more sustained manual attacks, more plies and interlayers are required to resist penetration. This resistance can be enhanced considerably by also incorporating polycarbonate plies in the laminated glass.

Laminated glass of sufficient thickness and interlayers can stop bullets. At around 70 mm thick and with an appropriate lay-up, armour-piercing rifle bullets can be prevented from penetrating. If polycarbonate sheets are built into the glass, thinner glass can serve the same

purpose. With bullets, there is a need to restrict spalling from the rear face, which can itself cause injuries. By making the back glass ply very thin (less than 2 mm thick), the amount of spalling can be restricted to a low level. To eliminate spall, a tough plastic film can be used on the non-attack side, which contains any spalled fragments.

It may easily be imagined, after reading the last two paragraphs, that very thick laminated glass is required to resist the forces of explosions. This would be the case for explosions where the source is close to the glass – in fact it is not cost-effective to use glass if this situation is foreseeable. Where there is a reasonable stand-off between the source of the explosion and the glass, the types of laminated glass which can contain the explosion forces are relatively thin: 7.5 mm (3 mm|1.52 mm|3 mm) laminated annealed glass in reasonable sizes, with a suitable interlayer, four-edge glazed, can resist significant explosion forces while staying in place. Indeed it may be a disadvantage to use glass which is too strong. Thin laminated glass with a thick polyvinyl butyral (PVB) interlayer (see section on *PVB (under laminated glass)* in Chapter 68) can absorb a large amount of the energy of the blast wave by deflecting, transferring only limited forces to the frame. A stronger, stiffer product may cause more significant damage by transferring high forces to the frame and building structure.

All of the security features of laminated glass (resistance to manual attack, bullets or explosions) are only effective if the glass is held in an adequate frame system designed specifically for the purpose. BS 5357 gives some limited advice on glazing of security glass. Most standard framing systems are inappropriate. It is strongly recommended that expert advice is sought if the glazing is to have any security function.

Decoration

Laminated glass can also be used with coloured or translucent interlayers. Coloured interlayers can be used for reducing light transmission and absorbing solar energy – an alternative to body-tinted solar control glass. Alternatively, the colours can be purely decorative. Translucent interlayers are used for obscuration or diffusion – they are not opaque and still let a proportion of the incident light through.

Laminating heat-treated glass

Laminating of heat-treated glass has some limitations, since most of the interlayers are supplied in sheet form and this only easily fills the gap between two panes if they are sufficiently flat. Toughened glass and heat-strengthened glass are not flat; they are susceptible to overall bow and also to roller wave. From a good-quality manufacturer, the distortion of the heat-treated glass can be controlled so that, using a 1.52 mm interlayer, laminated toughened glass, laminated heat-strengthened glass and laminate combinations of the two can be obtained.

Available sizes

Laminated glass in jumbo sizes (6000 mm × 3210 mm) made with PVB interlayers and float glass is readily available as stock glass in the commonly used thicknesses up to around 16.8 mm. For any other laminated glass thicknesses, glass types and interlayer variations there may be restrictions on what can be manufactured. These restrictions may be due to weight, size limits of the incorporated glass products, maximum size of interlayer sheet available, limitations on processing after lamination, etc. It is advisable to check whether the product can be manufactured if it is:

- more than 25.5 mm thick
- weighs more than 1000 kg per pane
- larger than 2400 mm × 4000 mm per pane
- incorporates a coating and is also thicker than 12 mm.

Coated glass to BS EN 1096

There is a large variety of types of coated glass from an ever increasing number of manufacturers. Many of these may be interchangeable in terms of their technical performance, but it is a mistake to consider mixing sources, as there will be colour variations between coated glass types which have effectively the same solar control and low E properties but are made by different manufacturers.

The currently available products can be generally categorised as shown in **Table 5**.

On-line coatings are sufficiently robust to be put through a laminating line and the coated glass can be toughened.

Some off-line coatings for solar control may also be sufficiently robust to be put through a laminating line, but the low E and high-performance coatings generally need to be applied to a pre-manufactured laminated glass if they are required in laminated form. This can limit availability (see section on *Coated laminated glass* in Chapter 68).

Coatings can be applied to tinted glass, as well as clear glass, extending the range of possible variations. Some off-line processors will also apply their coatings to enamelled glass.

In principle, the coating processes could be applied to any glass thickness, but there are practical restrictions. On-line coated glass is made in large runs (even 1-day production on a float line produces several hundred tons), so it is normally available only in a limited range of thicknesses. Sputter-coated glass has to pass through the 'gates' between the chambers of the process (see section on *Sputter coating* in Chapter 69), which limits the overall thickness of glass which can be coated. Many sputter-coating plants can only go up to 12 mm maximum overall thickness, while a few will go thicker. Heat-treated glass, may be distorted, may be further restricted as to the thicknesses that can be toughened.

In view of the very large range of possibilities for coated glass and the potential restrictions on available thicknesses

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Manufacturing process (see section on Coating in Chapter 68)	Function	Comment
On-line	Solar control	Medium performance at best. Some can be used on surface #1
	Low E	Emissivities around 0.16
	Self-cleaning	
	Low reflection	Single-sided product which needs to be laminated with the coating on both exposed surfaces to produce a low reflection glass
Off-line sputter coated	Solar control	Medium to good performance, but may have high visible reflection
	Low E	Emissivities from 0.1 down to 0.03
	High performance	Combined solar control and low emissivity. Most of these coatings are highly selective and are very efficient in terms of solar control while maximising light transmission
Off-line dip coated	Low reflection	

Table 5 Types of coated glass

and glass types which can be coated, the availability of any type of coated glass should be checked at an early stage in the specification for the building.

Many of the single-coated glass products are available in jumbo sizes (6000 mm × 3210 mm) on float glass or thin laminated float glass for use as stock. There may be size restrictions for the same coatings on heat treated glass types due to size limitations of the heat treatment processes.

Insulating glass units to BS EN 1279

The product standard for insulating glass units, BS EN 1279, assesses the properties and durability of the edge seals on insulating glass units. It does this by a series of tests to determine the mechanical properties and the ability of the edge seal to limit moisture vapour transfer to the interior of the cavities and to limit gas escape (for gas-filled insulating glass units).

Insulating glass units to BS EN 1279 can incorporate any glass type and thickness covered in this chapter. They may have any number of panes and cavities. In practice, if the overall thickness of an insulating glass unit is more than about 50 mm or if it has more than four panes and three cavities, automatic assembly lines may no longer be able to cope and availability will be restricted. Insulating glass units over 2500 mm × 4500 mm are also restricted in availability since there are few automatic lines which can cope with very large sizes and manual assembly of such large insulating glass units is difficult.

Fire-resistant glass to BS EN 357

The fire resistance of glass is classified by two functions. Any glass which is classified as fire resistant must be capable of resisting the passage of flames and smoke for a sufficient period, which is usually at least 30 minutes. Wired glass, borosilicate glass, alkaline earth glass and glass ceramics can all provide this function to varying extents. However, all these types of glass remain transparent in a fire and are relatively thin, so heat can be radiated and conducted through the glass which may possibly transmit the fire to adjacent combustible materials on the non-fire side. The laminated products with intumescent interlayers (see section on *Sodium silicate interlayer* in Chapter 68) not only stop the passage of flames and smoke but also go opaque and increase the thermal resistance of the glass to an extent sufficient, in some cases, to allow them to be classified as giving full insulation against fire.

Any fire-rated glass product will be given a classification such as ExIy to BS EN 357. E is integrity and x is the time in minutes for which the glass is capable of maintaining integrity (normally at least 30 minutes and higher in multiples of 15 minutes); I is insulation and y is the time in minutes for which the glass is capable of maintaining insulation (either 0 or at least 30 and higher in multiples of 15 minutes). If the glass does not have any insulation (I0) then the insulation classification may be omitted.

The ability of a glass product to be fire resistant depends not only on the glass but how it is glazed. A glazing system which enables one type of fire resistant glass to achieve its full potential may be no use when used for a different glass product. When fire-resistant glazing is considered, it is important to approach an installation company which specialises in this field and is appropriately accredited.

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Chapter 71

Characteristics and problems of glass products

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All glass products have properties and/or appearance that are not desirable. These properties may be variously called characteristics, effects, defects or problems, often depending on one's point of view (the point of view sometimes being figurative and sometimes literal).

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General glass defects

Since glass is made from materials which are dug out of the ground and, apart from washing, tipped straight into the melting tank, with materials added which are going to create bubbles, there will be occasional bubbles and stones (unmelted objects in the glass) in the finished product. The glass may also suffer from ream (slight variations in refractive index due to incomplete mixing of the raw materials), although this is rare in modern float glass. The optical quality or visual quality sections of the basic glass standard, BS EN 572, deal adequately with these particular issues, limiting the number and size of these faults in the various types of basic glass.

Since glass is transparent and used for looking through, any slight scratches are much easier to see than they are in other materials and may be obtrusive. It is inevitable that a few panes will be scratched during handling, in production, during transport and at installation. Such scratches are usually in small numbers.

Poor technique with a scraper blade, trying to clean off stubborn marks can generate large numbers of scratches. It is sometimes difficult to believe that glass is one of the most scratch-resistant of materials (**Figure 1**).

Glass has a reputation for extremely good durability, which does not immediately accord with the description of water attack in Chapter 67, section on *Glass and water*. Visible water attack does not usually result when the surface of the glass is open to the air, where it dries off regularly and when the quantities of water are relatively large, such as rain.

If the glass is left dirty for long periods, small quantities of water can get trapped under the dirt particles for longer periods and can cause visible surface attack (**Figure 2**). This is particularly the case when the dirt is itself alkaline, such as cement dust. If the dirt particles are themselves silicates, they can become welded to the glass surface.

For this reason, glass should be cleaned at regular intervals. The cleaning action removes deposits of dirt and also wipes off any early-stage visible water attack effects. Glass which is regularly cleaned will maintain its surface finish for many hundreds of years.

Annealed glass

Annealed glass is one of those materials which in normal service can be subjected to expansion and contraction strains significantly greater than its tensile breaking strain. In the case of annealed glass it is thermal expansion which is the problem.

The breaking strain under tension is typically between 0.03% and 0.15%. During its service life it may see a thermal expansion range up to 0.1%. If there is no constraint to the thermal expansion, then no stress is generated, but the natural situation of glass, with the central area exposed to solar radiation and the edge concealed in a frame so it can be much cooler leads to self-constraint and thermal stress (**Figure 3**). If a temperature difference of more than about 40 K can be generated between different parts of the glass then annealed glass is at risk of thermal stress breakage.

If annealed glass is considered as an option for part of the glazing, a thermal safety check should be performed if any of the following occur:

- the total solar absorption of the glass pane or combination of glass panes exceeds 40%;
- the glazing comprises three or more panes (in insulating glass units or secondary glazing);
- there are any backups (details of building structure or fittings that may trap heat behind the glass or reflect heat back into it);
- if blinds are used which can be closed so the gap between the blind and the glass is effectively unventilated (e.g. blackout-style blinds);
- if cavity blinds are installed.

Problems with thermal stress can be avoided by using toughened glass or heat strengthened glass, which are not at risk of thermal stress breakage at the temperature differences which may be generated in normal building use.

Figure 1 Scratches due to cleaning with a scraper blade

Clear float glass

Clear float glass has a light green tint to it due to the presence of small quantities of iron in the glass. Clear float glass from different suppliers may have different shades of green and/or different intensities of green. In most circumstances this is not obvious, but it can become noticeable if panes from different sources are viewed against a light background. Since clear float glass is a commodity product, it is possible to find glass from different origins in the same contract. **Figure 4**, for example, shows clear glass panes of different shades which are all within specification.

Figure 2 Water staining on glass which has been left uncleaned for too long

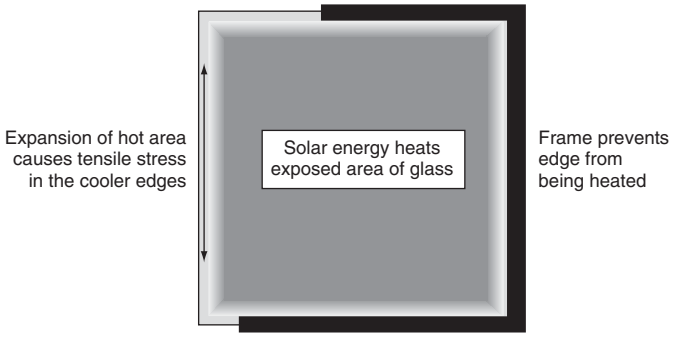

Figure 3 Development of thermal stress

Toughened glass and heat-strengthened glass

Roller wave and bow

During the heating process the glass is softened, which can lead to distortion of the glass from flat. This can produce an overall bow in the glass and also, for horizontally toughened glass, a wave-like distortion called roller wave. The standards BS EN 12150, BS EN 14179 and BS EN 1863

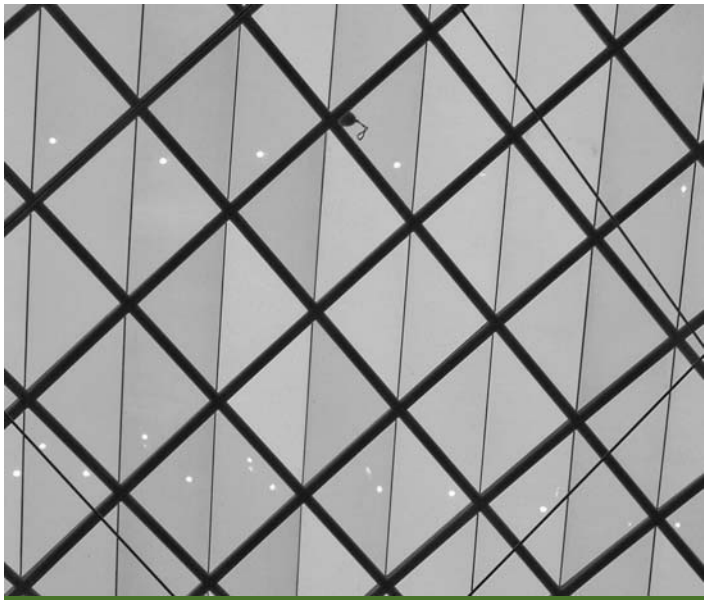

Figure 4 Clear glass in a roof with different green tints

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Float glass nominal thickness: mm	Roller wave maximum: mm
3, 4 and 5	0.5
6, 8 and 10	0.3
12, 15, 19 and 25	0.15

Table 1 Roller wave limits

prescribe limits for overall bow, but only mention that roller wave occurs, without any limitations.

Overall bow may contribute to distortion of visual reflections, but it is mainly associated with badly fitting frameless doors and misalignment of adjacent panes which are butted up against each other.

Roller wave can cause unsightly distortions of reflected images. The perceived distortion depends as much on the viewing aspect as the depth of the roller wave, appearing far worse when viewed from an angle, when observer and reflected object are both at some distance from the glass, or when the reflected object has straight lines.

In general, roller wave distortion of reflections is less obvious if the roller waves are horizontally aligned in the building. With modern toughening processes, it is relatively easy to keep the roller wave within limits. **Table 1** gives suggested limits. Individual manufacturers may be able to supply glass within tighter limits.

Roller wave is controlled by operating the plant with the glass temperature at the lower end of the viable processing range (600–640°C). Enamelled glass may need to be processed towards the upper end of this range in order to fire the enamel into the glass correctly, so it is likely to have more accentuated roller wave than clear glass.

Anisotropy

The visual effect called anisotropy, also known as iridescence, bi-refringence, toughening marks, quench marks or leopard spots, arises from local variations in the toughening stress. The variations occur because the cooling air is blown at the glass through an array of nozzles during the quench part of the process. Immediately under a nozzle the cooling effect and stress is greater than away from the nozzles. If the glass is static in the quench, then spots of greater stress are formed. If the glass or the quench head is moving then the spots are smeared. These local variations in toughening stress are present in every pane of toughened glass.

Glass has a natural property which twists polarised light when the glass is stressed, so the slight variations in toughening stress may be visible when the glass is viewed at an angle (see **Figure 5**) in polarised light. Since light can be polarised by a variety of effects – reflection, blue sky (especially in the evening), and hazy sky or light cloud, for example – they will show up from time to time. Viewing the glass using a polarising filter enhances the effect.

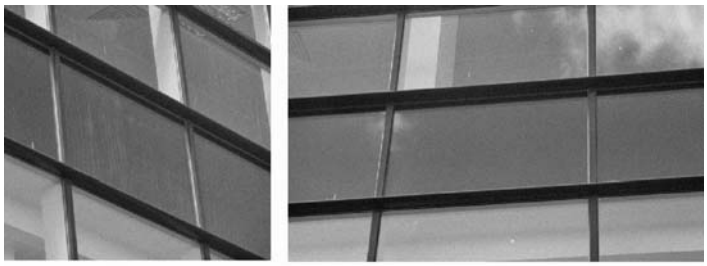

Figure 5 Anisotropy visible when (a) the glass is viewed at an angle, or (b) not when viewed from directly in front

The pattern of the anisotropy is determined by the mechanics of the toughening plant (usually not in the control of the operators) and can vary from distinct spots (as are sometimes visible in car rear windows), to linearly smeared spots to smeared areas.

Anisotropy, which is mentioned in BS EN 12150, BS EN 14179 and BS EN 1863, is a natural property of toughened glass and cannot be eliminated or corrected in any way.

Roller marks, roller pick-up and roller haze

As the soft glass passes over the rollers towards the end of the heating cycle, any dust or deposits on the rollers will make an impression in the glass. Sometimes this will stick in the glass and this has given rise to the term roller pick-up. Local hot spots from contact with the rollers may cause the glass surface to 'dimple'. These effects occur more often if the glass is heavy or in the heating oven for longer. Thicker glass is both heavier and needs a longer heating time than thinner glass, so it is more likely to get roller marks. Roller marks will be present to some extent on all toughened glass or heat-strengthened glass 12 mm or thicker and, according to BS EN 12150, BS EN 14179 and BS EN 1863, may be present on any glass 8 mm or thicker.

Most roller marks are individually small and are only rarely visible or obtrusive when the glass is viewed from a reasonable distance. They can sometimes be clustered close to the edge if the glass has arched off the rollers during the heating process. If the glass is generally overheated, the number of roller marks can cause an effect known as 'orange peel' where the appearance of objects viewed through the glass is blurred. Orange peel is simply due to poor process control.

Roller haze is a band of very fine scratches down the centre of a pane of glass caused by a pane, which has curled upwards during the heating process, sliding on the rollers during the reversal of the roller motion. It appears as a misty band in certain lighting conditions. Roller haze is very shallow and is easily polished out.

Coated glass

All coated glass has different reflected colour depending on which side it is viewed from, so if some is installed a different way around from others, this may be noticeable. If there is no colour or shade difference looking through the glass, then wrong orientation is likely to be the cause of any difference in reflected colour or shade.

In all the coating layers, there is a tolerance on the thickness laid down. Since the colour of the coated glass is largely generated by the thicknesses of the various layers, it follows that there will be a tolerance in terms of colour. This tolerance is called a 'colour box'. The colour variation occasionally becomes visually obvious. The eye is most sensitive to complete changes of colour, so coated glass which is very close to neutral in colour is more likely to give obvious colour variations.

The same coated glass made at different factories, or on different batches from the same factory, may have colour differences while still being within the colour tolerance.

Laminated glass

Delamination

Delamination is caused by three basic reasons:

- incompatibility of the interlayer with the glass surface
- chemical contamination
- stress.

The interlayers used for laminating glass are formulated to have adhesion to the glass surface. If the glass is coated, the adhesion of the interlayer material to the coating may not be as good, making the laminated glass more susceptible to delamination. If laminating a coated glass with the coating against an interlayer is being considered, the compatibility needs to be checked.

If the glass is not properly cleaned before laminating, there may be some contamination of the surface, leading to either poor adhesion or a chemical reaction between the contaminant and the interlayer material. Either of these could develop into delamination. The exposed edges of the interlayer are vulnerable to external sources of chemicals, so glazing materials need to be compatible with the interlayer. Water is one chemical which will cause delamination, so exposed edges of interlayers in damp environments may experience local delamination. The temptation to ask for exposed edges to have a protective material applied should be treated with caution – most of the materials which may be considered for resisting water contain solvents, many of which are far more detrimental to the interlayer than water.

Mechanical stress on the interlayer could come from a number of sources. Laminated heat-treated glass, which is distorted, is prone to delamination if the interlayer does not adequately fill parts of the gaps between the panes. The stress in the interlayer may be sufficient for it to delaminate, particularly from an edge where contaminants may initiate delamination. A 'sunburst' delamination is symptomatic of stress delamination.

Large areas of delamination suggest there is a problem with either the individual pane or with that particular laminate composite. Panes with large areas of delamination should be replaced, preferably after the reason for the delamination has been determined and corrective action undertaken if necessary.

There is a current school of thought which believes that it is possible to completely avoid edge delamination in laminated glass. This is not the case. All current laminated glass interlayers can develop small amounts of delamination or bubbles in the interlayer at the edges of the glass panes despite the best efforts of those involved in manufacture and installation.

Small levels of delamination, up to 6 mm into the glass, are unlikely to be significant in terms of the strength, post-breakage characteristics, or safe breakage properties of the laminated glass. Depending on the glazing system, they may be considered as visual faults if the edges of the laminated glass are unframed. In external glazing (e.g. canopies), the delaminated areas may harbour debris and/or algae and eventually turn green/brown.

Since edge delamination or edge bubbles will occur in some laminated glass panes, it should be considered whether visible edges are appropriate on a contract or whether a simple edge channel should be applied to conceal them.

Shear transfer

The materials used for the interlayers of laminated glass are much less stiff than the glass and also have stiffness and strength that vary both with temperature and the duration of any applied forces. The interlaminar shear stress developed when loads are applied to laminated glass depends on the amount and type of load, the size of the piece of laminated glass and the way in which it is glazed. Given all the above variables, the reaction of a particular piece of laminated glass when subjected to loads may vary from being approximately equivalent to a similar thickness of monolithic glass to being approximately equivalent to a collection of unadhered glass plies.

In respect of short-duration loads, where some shear transfer may be expected, the way in which laminated glass is treated varies from country to country. In BS 6262-3, for example, the wind load charts for four edge glazed laminated glass assume full shear transfer, i.e. that it acts like the equivalent monolithic glass. Most other European countries do not consider full shear transfer can be possible. The proposed European standard

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Figure 6 Delamination due to chemical contamination

for calculating glass strength, prEN 13474, will assume partial shear transfer for short-duration loads.

In respect of long-duration loads, e.g. self weight, glass shelves, glass floors, the general consensus is that no shear transfer should be considered.

Insulating glass units

Effects due the presence of the cavity gas

Insulating glass units have a fixed quantity of air or other gas sealed into them at the time of manufacture. This will be at the factory temperature and at the atmospheric pressure at the time of sealing. The presence of this fixed quantity of gas has two specific effects which need to be considered.

Load sharing

The pressure of the air/gas inside the cavity is approximately 100 kN/m^2. If a wind or snow load is applied to one side of the unit, it deflects the glass and may cause the cavity to compress slightly. Any such compression increases the pressure inside the cavity, which counters the applied load and also causes the opposite pane to deflect. Considering wind or snow loads, which rarely exceed 3 kN/m^2, the maximum compression of the cavity can be no more than 3%. This means that both panes must deflect the same amount, to within considerably less than 1 millimetre error. They can only do this if the panes share the applied load between them in proportion to the cube of their thicknesses. This occurs even if it is the thinner pane which has the load applied.

When the applied loads are not uniformly distributed, the principle of load sharing still applies, but because the loaded pane is subjected to concentrated loads and the other pane is subjected to a uniformly distributed load from the cavity gas, the deflection profiles and volume displacements will be different. This means that the loaded pane takes more than the share of the applied load indicated above, but it still obtains some support from the other pane.

Bowing and dishing

During its subsequent service life, the temperature of the gas inside the unit will vary and the atmospheric pressure will also vary. The gas/air inside the cavity will expand and contract with these variations. In the UK, manufacturing temperature is higher than average service temperature of the cavity gas, so in the UK, insulating glass units are dished inwards more often than they are bowed outwards. Considering the possible ranges of temperature and atmospheric pressure, the width of the cavity in the centre of the insulating glass unit may be as low as 50% of the nominal cavity width and as high as 130% of the nominal cavity width without there being any fault in manufacture. Dished insulating glass units cause interesting distortions of reflections (**Figure 7**).

If the units are of a relatively small size, such that the smaller dimension of the pane is around 70 times the glass thickness, then the stress generated by natural temperature and pressure effects bowing or dishing the glass may be sufficient to crack annealed glass. This effect is exacerbated if the insulating glass unit has one pane thicker than the other, since the deflection and stress is mainly in the thinner pane.

The stresses and deflections in the glass and the insulating glass unit edge seal developed by this phenomenon are directly proportional to the total of the cavity widths in an insulating glass unit. This places an effective limitation on the total of the cavity widths of around 30 mm.

Optical interference

Newton's rings

If an insulating glass unit is manufactured with too little air or other gas in the cavity, it is possible that the two panes

Figure 7 Distorted reflections from dished insulating glass units

may touch in the centre of the insulating glass unit. This results in a series of concentric quasi-circular coloured interference fringes, which are known as Newton's rings. If Newton's rings are present then the insulating glass unit has been badly manufactured.

Brewster's fringes

If the glass thicknesses of the two panes in an insulating glass unit are very similar (within a few wavelengths of light), then an interference effect can arise, due to the reflections from the four glass surfaces in the insulating glass unit interfering and causing a rainbow of interference fringes visible in reflection. This used to be a common occurrence in the 1970s, when the insulating glass units were regularly made with two panes of float glass cut from next to each other. The phenomenon is much less likely to occur when the two panes are different products manufactured at different times, as is normally the case with modern insulating glass units. Brewster's fringes are not a fault.

Sound insulation

It is a common misconception that the sound insulation of insulating glass units is better than that of single glass. This is not the case, particularly if the sound source is traffic noise. **Table 2** gives some examples of road traffic sound reduction taken from BS 6262-2.

Glass	Traffic sound reduction: dB
6 mm single glass	28
6 mm/12 mm/6 mm insulating glass unit	27
12 mm single glass	32

Table 2 Example of road traffic sound reduction taken from BS 6262-2

The sound reduction of glass products is improved by approximately 4 dB when the weight of glass is doubled. If the same total weight of glass is in separate panes which are close together, resonances between them cause additional sound to be transmitted, which can offset any increase due to the weight. The resonances only disappear when the gap between the panes is in excess of 50 mm to 75 mm, which is secondary glazing rather than insulating glass units. If one of the main requirements of any glazing is sound insulation (e.g. in office partitions), care should be taken over the selection of the glass.

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British Standards relating to glass in buildings

BS EN ISO 12543	Glass in Building – Laminated Glass and Laminated Safety Glass
BS EN ISO 12543-1	Definitions and Description of Component Parts
BS EN ISO 12543-2	Laminated Safety Glass
BS EN ISO 12543-3	Laminated Glass
BS EN ISO 12543-4	Test Methods for Durability
BS EN ISO 12543-5	Dimensions and Edges Finishing
BS EN ISO 12543-6	Appearance
BS EN ISO 14438	Glass in Building – Determination of Energy Balance Value – Calculation Method
BS EN ISO 14439	Glass in Building – Glazing Requirements – Glazing Blocks
BS EN 356	Glass in Building – Security Glazing – Testing and Classification of Resistance Against Manual Attack
BS EN 357	Glass in Building – Fire Resistant Glazed Elements with Transparent or Translucent Glass Products – Classification of Fire Resistance
BS EN 410	Glass in Building – Determination of Luminous and Solar Characteristics of Glazing
BS EN 572	Glass in Building – Basic Soda Lime Silicate Glass Products
BS EN 572-1	Definitions and General Physical and Mechanical Properties
BS EN 572-2	Float Glass
BS EN 572-3	Polished Wired Glass
BS EN 572-4	Drawn Sheet Glass
BS EN 572-5	Patterned Glass
BS EN 572-6	Wired Patterned Glass
BS EN 572-7	Wired or Unwired Channel Shaped Glass
BS EN 572-8	Supplied and Final Cut Sizes
BS EN 572-9	Evaluation of Conformity/Product Standard
BS EN 673	Glass in Building – Determination of Thermal Transmittance (U Value) – Calculation Method
BS EN 674	Glass in Building – Determination of Thermal Transmittance (U Value) – Guarded Hot Plate Method
BS EN 675	Glass in Building – Determination of Thermal Transmittance (U Value) – Heat Flow Meter Method
BS EN 1036	Glass in Building – Mirrors from Silver Coated Float Glass for Internal Use
BS EN 1036-1	Definition and Description
BS EN 1036-2	Evaluation of Conformity/Product Standard
BS EN 1051	Glass in Building – Glass Blocks and Paver Units
BS EN 1051-1	Definition and Description
BS EN 1051-2	Evaluation of Conformity/Product Standard
BS EN 1063	Glass in Building – Security Glazing – Testing and Classification of Resistance Against Bullet Attack
BS EN 1096	Glass in Building – Coated Glass
BS EN 1096-1	Definitions and Classification
BS EN 1096-2	Requirements and Test Methods for Durability of A, B and S Coatings
BS EN 1096-3	Requirements and Test Methods for Durability of C and D Coatings
BS EN 1096-4	Evaluation of Conformity/Product Standard
BS EN 1098	Glass in Building – Determination of Thermal Transmittance (U Value) – Calibrated and Guarded Hot Box Method
BS EN 1279	Glass in Building – Insulating Glass Units
BS EN 1279-1	Generalities and Dimensional Tolerances and Rules for the System Description
BS EN 1279-2	Long Term Test Method and Requirements on Moisture Vapour Penetration
BS EN 1279-3	Long Term Test Method and Requirements for Gas Leakage Rate and for Gas Concentration Tolerances

BS EN 1279-4	Methods of Test for the Physical Attributes of Edge Seals
BS EN 1279-5	Evaluation of Conformity/Product Standard
BS EN 1279-6	Factory Production Control and Periodic Tests
BS EN 1288	Glass in Building – Determination of the Bending Strength of Glass
BS EN 1288-1	Fundamentals of Testing Glass
BS EN 1288-2	Coaxial Double Ring Test on Flat Specimens with Large Test Surface Areas
BS EN 1288-3	Test with Specimen Supported at Two Points (Four Point Bending)
BS EN 1288-4	Testing of Channel Shaped Glass
BS EN 1288-5	Coaxial Double Ring Test on Flat Specimens with Small Test Surface Areas
BS EN 1748-1	Glass in Building – Special Basic Borosilicate Glass Products
BS EN 1748-1-1	Definitions and General Physical and Mechanical Properties
BS EN 1748-1-2	Evaluation of Conformity/Product Standard
BS EN 1748-2	Glass in Building – Special Basic Glass Ceramics Products
BS EN 1748-1-1	Definitions and General Physical and Mechanical Properties
BS EN 1748-1-2	Evaluation of Conformity/Product Standard
BS EN 1863	Glass in Building – Heat Strengthened Soda Lime Silicate Glass
BS EN 1863-1	Definition and Description
BS EN 1863-2	Evaluation of Conformity/Product Standard
BS EN 12337	Glass in Building – Chemically Strengthened Soda Lime Silicate Glass
BS EN 12337-1	Definition and Description
BS EN 12337-2	Evaluation of Conformity/Product Standard
BS EN 12150	Glass in Building – Thermally Toughened Soda Lime Silicate Safety Glass
BS EN 12150-1	Definition and Description
BS EN 12150-2	Evaluation of Conformity/Product Standard
BS EN 12488	Glass in Building – Assembly Rules – Glazing Systems and Glazing Requirements
BS EN 12600	Glass in Building – Pendulum Test – Impact Test Method for Flat Glass and Performance Requirements
BS EN 12603	Glass in Building – Determination of the Bending Strength of Glass – Procedures for Goodness of Fit and Confidence Levels for Weibull Distributed Data
BS EN 12758	Glass in Building – Glazing and Airborne Sound Insulation – Product Descriptions and Determination of Properties
BS EN 12898	Glass in Building – Determination of the Emissivity
BS EN 13022	Glass in Building – Structural Sealant Glazing
BS EN 13022-1	Glass Products for Structural Sealant Glazing Systems – Supported and Unsupported Monolithic and Multiple Glazing
BS EN 13022-2	Assembly Rules
BS EN 13024	Glass in Building – Thermally Toughened Borosilicate Safety Glass
BS EN 13024-1	Definition and Description
BS EN 13024-2	Evaluation of Conformity/Product Standard
prEN 13474	Glass in Building – Determination of the Strength of Glass Panes
prEN 13474-1	Glass and Glass Products for Fenestration
prEN 13474-3	General Method of Calculation and Determination of Strength of Glass by Testing
BS EN 13541	Glass in Building – Specification for Explosion Pressure Resistant Security Glazing – Classification and Test Method
BS EN 14178	Glass in Building – Basic Alkaline Earth Silicate Glass Products
BS EN 14178-1	Float Glass
BS EN 14178-2	Evaluation of Conformity/Product Standard
BS EN 14179	Glass in Building – Heat Soaked Thermally Toughened Soda Lime Silicate Safety Glass

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BS EN 14179-1	Definition and Description
BS EN 14179-2	Evaluation of Conformity/Product Standard
BS EN 14321	Glass in Building – Thermally Toughened Alkaline Earth Silicate Safety Glass
BS EN 14321-1	Definition and Description
BS EN 14321-2	Evaluation of Conformity/Product Standard
BS EN 14449	Glass in Building – Laminated Glass and Laminated Safety Glass – Evaluation of Conformity/Product Standard
BS 952	Glass for Glazing
BS 952-1	Classification
BS 952-2	Terminology for Work on Glass
BS 5357	Code of Practice for Installation of Security Glazing
BS 5516	Patent Glazing and Sloping Glazing for Buildings
BS 5516-1	Patent Glazing
BS 5516-2	Sloping Glazing
BS 6180	Code of Practice for Barriers in and about Buildings
BS 6206	Specification for Impact Performance Requirements for Flat Safety Glass and Safety Plastics for Use in Buildings (Superseded by BS EN 12600)
BS 6262	Code of Practice for Glazing for Buildings
BS 6262-1	General Methodology for the Selection of Glazing
BS 6262-2	Energy, Light and Sound
BS 6262-3	Fire, Security and Wind Loading
BS 6262-5	Frame Design Considerations
BS 6262-4	Safety Related to Human Impact
BS 6262-6	Special Applications
BS 6262-7	Provision of Information
BS 6993	Thermal and Radiometric Properties of Glazing
BS 6993-1	Method for Calculation of the Steady State U Value (Thermal Transmittance)
BS 8000	Workmanship on Building Sites
BS 8000-7	Code of Practice for Glazing
BS 8233	Code of Practice for Sound Insulation and Sound Reduction for Buildings

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Section 10: Non-conventional materials

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Chapter 72

Low-carbon natural building materials

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This chapter reviews the modern use of alternative, non-conventional, low-carbon materials in building construction, including unfired clay, lime, hemp-lime and other crop- and plant-based resources. The objective is to provide an introduction to current knowledge and practice, establishing the basis for further investigation by the reader.

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Introduction

Natural materials, such as stone, earth and timber, have provided the basis for human infrastructure development throughout civilisation. Many so-called vernacular techniques rely on natural and locally processed materials. However, the industrial revolution displaced many natural technologies for manufactured materials, including iron, steel, concrete and plastics. Concerns over the environmental impacts of building and infrastructure over the past 20–30 years have seen a renaissance in the interest of many traditional technologies, as they can offer lower carbon approaches to building. However, many natural materials, such as earth and plants, offer other benefits, including hygrothermal regulation of internal spaces. As well as minimising transport impacts, the greater use of local materials, such as cob or stone, maintains a regional variation in architecture often lost to the modernist ubiquity of concrete or steel.

Natural materials can be broadly divided as either inorganic (stone based) or organic (plant based). Although subject to processing in conversion from the resource to the building, natural materials often maintain their original chemical form. Lime, formed from burning mineral sources of calcium carbonate, is an exception to this. The inclusion of lime under natural building materials is, however, justified as it is often widely used and the most suitable bonding and finishing material for a variety of natural building materials.

Many traditional natural building materials, such as clay, hemp and straw, are hygroscopic. The materials alter their moisture contents in response to changing environmental conditions, always seeking to reach equilibrium with the ambient conditions. As surrounding humidity levels increase, clay materials absorb water vapour, reducing increases in the surrounding humidity levels. Similarly, when ambient air humidity levels are low, clay materials will release moisture into the atmosphere. As environmental conditions are in a state of change, this hygroscopic behaviour is an on-going dynamic process, buffering ambient internal relative humidity levels to around 40–60%. This has potential indirect health benefits for building occupants, especially asthma sufferers, and removes condensation problems. Clay materials have been used to limit relative humidity in storage areas, such as libraries or archives. Plant-based materials, such as hemp-lime, exhibit similar properties. However, hygroscopic behaviour is only realised if finishes applied to walls are vapour permeable (breathable). These include lime- and clay-based plasters. Hygroscopic behaviour also improves building environmental behaviour. As materials release or absorb moisture in response to environmental changes, latent heat energy is required. This provides the material with additional thermal inertia or thermal mass, delaying responses to changes in environmental conditions.

As well as passive environmental control and improved health, other benefits of using natural materials can include lower embodied carbon content, increased potential for recycling, greater use of local materials, reduction in reliance on non-renewable materials and opening new markets for agricultural products. Drawbacks and challenges to wider use of natural traditional materials in modern construction include frequent reliance on traditional in-situ based construction, low strength and poor seismic resistance, higher initial cost of construction and uncertainties over durability in novel applications.

This chapter briefly outlines properties, production and construction using unfired clay, including rammed earth and adobe, lime, hemp-lime composites and briefly other crop-based materials in construction, some of which are covered in greater detail elsewhere in this book. The advantages and drawbacks of each material are also briefly considered. The chapter will hopefully encourage designers and builders, to seek further information and use materials in a wide variety of projects.

Earth as a building material

Introduction

In civil works, sub-soil (earth) is the material for geotechnical engineering, used to form many features, including cuttings, embankments, causeways, dams, retaining walls and foundations. However, the engineering study of sub-soils has, until comparatively recently (Walker *et al.*, 2005; Jaquin, 2008), rarely extended into their widespread historic use in earthen buildings.

Earthen architecture has a very long vernacular tradition throughout civilisation (Minke, 2006). Pre-industrialised society earth has often been the building material of necessity rather than choice. The absence of affordable alternatives, combined with its widespread availability and low cost, has seen it used successfully around the world. However, earth building continues to have a poor image, due its association with impoverished rural communities, perceived lack of modernity and, perhaps most significantly, its low resistance to natural disasters (earthquakes, floods, etc.). Although the greatest use of earth continues to be in providing millions of simple houses, some notable architectural feats have been accomplished using earth. These include large sections of the Great Wall of China, the Great Mosque of Djenne in Mali (**Figure 1**), the Alhambra in Spain, chateaux in France and multi-storey adobe tower blocks in Yemen.

Earthen buildings are found throughout Europe. There are an estimated 500 000 earthen buildings occupied in the UK, with notable regional concentrations in south-west England, East Anglia, Cumbria and Buckinghamshire (Keefe, 2005). Other western European countries, such as Germany, France, Italy, Spain and Portugal, have similar or even higher proportions of historical earth buildings (Minke, 2006). Most of these existent stock have origins prior to the last century and, despite efforts to promote earth building after both world wars, in periods of acute material shortages earth building has largely been displaced by other mass-produced materials (Williams-Ellis, 1999).

Figure 1 Great Mosque, Djenne, Mali

Building with earth comprises using natural sub-soils, often together with other materials, to build walls, roofs and floors of buildings and other structures. A wide variety of techniques has developed through vernacular use, falling broadly between wet mud methods, such as adobe and cob, and dry compacted methods, including rammed earth and compressed earth blocks. Methods reflect the variations in quality of raw materials and regional constraints, including access to other resources and climate.

Vernacular earth building developed in response to the characteristics of the material. Most notably, walls are solid, thick with comparatively few openings. This has provided buildings with high thermal mass that provide comfortable living spaces in a range of climates. Thick walls reflect low strength of the material and its susceptibility to loss of strength in contact with water. The use of natural soils, with very little processing, offers a building material with very low environmental impact. This has been a primary driver in the renaissance of earth as a modern building material in the UK. Traditional techniques, such as rammed earth, have been recently used to build highly desirable, and expensive, modern buildings. However, it is unlikely that vernacular techniques, such as cob and rammed earth, will have a significant impact on future building practice in developed, high labour cost, economies. Modern developments, such as extruded bricks and panels, plasters and mortars and future innovations will, however, continue to provide new opportunities and markets for unfired clay buildings.

Materials

Soils

Most earth building is comprised of natural sub-soils. In the simplest cases these can be found from site excavations, or sourced from local borrow pits, quarries or, increasingly, directly from suppliers of prepared unfired clay materials. Clay fines within the sub-soil provide the cohesive binder to fix the sand and coarser aggregate matrix. Analogies to concrete have been made (Anger *et al.*, 2008). However, unlike concrete, the clay suction that binds the material together is much weaker than cement paste, and varies in response to changes in material moisture content. Pore suction pressures develop as the material dries out; earth materials behave as unsaturated soils. Final material properties are dependent on the interplay of many factors, including initial processing of materials, level of compaction provided, rate and extent of drying, and stress history. Testing of materials needs therefore to be representative of the expected as-built conditions.

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Traditionally, soils for building were selected by experienced craft builders. Regional distributions of earth building are largely a reflection of the success with the materials found in these areas. In many cases this process continues today. However, where specification is appropriate, selection of sub-soils for earth building rely on engineering classification tests, including grading and clay plasticity (Walker *et al*., 2005). Specifying grading limits and relative proportions of clay, silt and gravel are helpful, the most reliable means of material selection is through performance testing, traditionally through trial and error and today often through laboratory testing and trial walls. Earth building is remarkably tolerant to variations in sub-soil properties when used by experienced workers, so no single material grading specification for rammed earth or adobe materials is appropriate as it risks both including unsuitable and excluding suitable materials. There should be sufficient clay to bind the materials together, perhaps as little as 5% by mass, without undue shrinkage on drying. The gravel and sand content is often the highest proportion of materials. A well-graded material will achieve higher density when compact, generally developing higher strengths and resistance to weathering. Rammed earth walls exposed to weathering rely on a relatively high gravel content to resist erosion, whilst adobe blocks can be made with materials with little or no aggregates. Earth plasters are mostly comprised of fine sand, silt and clay.

It is perhaps easier to define what soil characteristics are least desirable. Highly active clays, such as montmorillonite, and to a lesser extent illite, are problematic. Their high plasticity and movement in response to changes in moisture content make them difficult, but not impossible, to work and more prone to water damage. Organic matter, for example straw, is commonly used as an additive (see below). However, highly organic soils, including top soils, are best avoided due to likely break down of organic matter (and their high horticultural value). Some soils contain dispersive soluble salts, leading to their rapid breakdown on contact with water. Soils with high levels of natural or industrial contaminants, such as heavy metals, are also to be avoided. Poorly or gap graded sands and aggregates are unsuitable alone, but with the addition of other materials can form the basis for a suitable material.

Natural sub-soils as a resource for building are enormously varied in distribution, qualities and characteristics. These depend on complex geological processes that have given rise to their present formation. Soil deposits on site often vary with stratigraphy. When selecting materials it is important to identify that the chosen source has sufficient quantity of consistent material. Modern site investigation methods can be used effectively to identify materials.

Recent rammed earth projects in the UK have progressed on the approach of re-using on-site material in the construction of walls. This minimises the environmental impact of materials processing and transportation but introduces uncertainty concerning suitability. Vernacular approaches to soil selection are often not suited to modern construction projects, reliant on contracts and performance specifications. Testing of materials is essential in these circumstances to assess suitability of material through soils testing (grading, clay plasticity, etc.) and performance testing materials. Where in-situ materials are not ideal, engineered blends can be developed through the addition of locally sourced materials, though this requires careful mixing and preparation on site. The expense and uncertainty of testing has led to development, especially in Germany, of prepared materials from a central supplier. For example, Claytec Ltd supplies bags of rammed earth material to order. Cement stabilisation offers an alternative approach to addressing the uncertainty of materials but is likely to increase the environmental impact of construction.

Additives

Throughout the development of earth building, a wide variety of natural materials has been added to improve the qualities of sub-soil. The most well known is chopped straw (Minke, 2006; Keefe, 2005). Cereal straw fibres reinforce materials to reduce shrinkage cracking on drying. This is most commonly used in adobe, cob, plasters (more finely chopped straw) and other forms of wet earth construction. Straw content is typically between 1 and 3% of soil mass, depending on soil properties (clay content and type, initial moisture content). As well as straw, other plants have been used, including reeds, finer fibres of hemp, flax or sisal, and leaves or twigs. Fibres provide little significant increase in flexural strength, rather controlling shrinkage and offering post-cracking ductility. Other well-known additives to sub-soils include a variety of animal products, including cow manure (fibres + enzymes), hair, casein (often combined with lime), blood and urine. These are often used to improve wearing and water resistance of earth floors and external plasters.

Lime has been used as a stabiliser in earth construction for many years. Many Spanish rammed earth buildings, including the Alhambra, have used lime to enhance strength and weather resistance (Jaquin, 2008). Lime binders, in the form of calcium hydroxide, can react with clay minerals to form weak hydraulic bound material as well as gaining strength through carbonation. Given the historic success of lime in stabilising sub-soils it is not surprising that during the twentieth century cement stabilisation of earth materials also became well established. The addition of cement can provide a significant improvement in strength and resistance to water. Cement addition varies between 4% and 15% by mass, possibly leading to far greater cement use than competing methods such as concrete block masonry. Cement stabilisation is most effective

without clay present in the matrix; clay levels are therefore limited to the minimum necessary for construction. For some people promoting earth building, however, the addition of cement is akin to the production of poor-quality concrete, using aggregates falling outside normal concrete specifications and increasing the carbon footprint of the material. To those who promote cement stabilisation, however, its use improves reliability and consistency of material, removing many of the concerns associated with a material susceptible to water deterioration. On balance an over-reliance on cement stabilisation is to be avoided if the full advantages of earthen materials are to be realised. However, its use as a stabiliser should be considered appropriate and prudent, for example where enhanced strength and durability is necessary or where transportation of bulk materials can be avoided.

Other materials used to improve soils have often been adopted from pavement construction and include sodium silicate and lime. Although little used in the UK, bitumen emulsion is used in the USA and elsewhere as a waterproofing agent (McHenry, 1989). Gypsum has also been used to control shrinkage in poured earth; waste gypsum plasterboard has been used as a filler material in extruded clay blocks.

The addition of crop-based materials can reduce the carbon footprint of clay materials through capture and storage of carbon during the plant growth. This benefit depends on secondary carbon used in harvesting, processing and transportation. The addition of lime and cement will significantly increase the embodied carbon, but with benefit of improved durability and strength.

Processing

As well as by addition of materials, sub-soils can be improved during processing and placement. Compaction during construction or fabrication of blocks is an effective means of improving materials. Densification, by removing air, improves strength and durability, though reduces thermal resistance. Many earth construction techniques use some form of densification. Mixing of materials improves the quality and properties of soils, especially where the natural material is heterogeneous or two or more materials are combined. The uniform dispersion of materials, especially binders, will improve properties (Minke, 2006). Modern clay working uses extrusion processes. As well as compacting materials as they are driven through the die, extrusion aligns fibres within the material.

Construction techniques

Traditional earth construction methods, of which there are many, can be broadly classified as either wet (moulded) or dry (compacted) methods. Adobe and cob are examples of wet, while rammed earth is an example of a dry compacted method. The initial moisture content of materials during preparation is the most obvious difference in approach, often reflecting differences in properties of the materials used. Wet methods of construction commonly use materials with higher clay contents than the drier methods of construction. Heavy clay soils are much easier to work and mix once plastic or liquid. Once sufficiently wet, materials become much easier to mix, mould and work into the required forms. Controlling shrinkage on drying is a primary concern, hence the widespread use of fibres. The level of water addition depends on qualities of material and varies with method of construction. Compaction, the removal of air from the matrix, is still an important element.

There are many forms of earth construction (Houben and Guillaud, 1994), with regional and national variations according to soil types, climate and culture. The most common types found and used in future in the UK are outlined below. Others missing from this list include: poured earth, wattle and daub, dug-out, earth roofs, sod and cut blocks, in-fill (earthships), direct shaping.

- Rammed earth: a traditional form of construction, sub-soil is compacted at or close to its optimum moisture content within the formwork (Walker *et al.*, 2005). The soil is compacted progressively in layers between 100–150 mm thick. Traditionally, compaction was undertaken manually, but in the UK pneumatic compactors are more commonly employed. Formwork needs to have strength and stiffness to withstand compaction pressures. Various forms have been employed. In modern works, concrete forms are preferred (**Figure 2**). Walls are typically 300 mm thick or more depending on requirements. It is possible to ram sections less than 100 mm thick, although below 300 mm thick is increasingly difficult and requires formwork without through-ties. Traditionally, walls are solid. The aesthetic qualities of rammed earth, possible through the compaction process, have greatly assisted its reintroduction in modern works. Soil moisture at compaction needs to be care-

Figure 2 Concreting formwork system in use for rammed earth

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Figure 3 Manual earth block press

Figure 4 Mechanised pressed earth block production

fully controlled to the optimum moisture content; traditionally determined by the drop test, but often now initially established using the Proctor test (BS 1377-4: 1990).

- Compressed earth blocks: the development of compressed earth blocks is attributed to the development of the manual CINVA ram in Colombia in the 1950s, although compaction of moist, rather than wet soil is of course much older. Subsequently, many variations of the CINVA ram have been developed, primarily for applications in the majority world (**Figure 3**). Loose prepared soil is placed in a mould and compacted by manual leverage or hydraulically. High levels of compactive effort can be achieved with the simplest presses, offering dense, strong blocks with excellent dimensional consistency. Solid blocks are often laid as concrete blocks, with 10 mm mortar joints, although the dimensional consistency would often allow much thinner joints or even dry-stacked mortarless construction. Compaction greatly improves strength. Compressed earth blocks are often stabilised with cement, although through appropriate selection of materials this would not be necessary. Curing and drying of blocks prior to construction ensures that shrinkage of materials takes place before construction. The similarity to concrete blocks, compressed earth blocks typically around $300 \times 140 \times 90$ mm high, has allowed greater cultural uptake. Although compressed earth block technology has developed in Asia and Africa, the high cost of labour has limited wider uptake in the UK. In the US, and elsewhere, labour costs have led to development of mechanised presses capable of producing a few thousand blocks per day, compared to the few hundred produced by a small team using a manual press (**Figure 4**). Compressed earth block masonry is generally designed, detailed and constructed as solid concrete blocks.
- Adobe: the use of sun-dried mud bricks dates back to the origins of human settlement and civilisation. In its simplest form, adobes are sun-dried masonry blocks, formed from plastic moulding by hand, wet sub-soil material. In many parts of the world this technique has changed little over millennia (**Figure 5**). Sub-soil is mixed with water to the appropriate consistency. Blocks are often formed within a timber mould on the ground and allowed to gain strength by drying in the sun. This activity is reliant on a consistently dry season, although a form of adobe construction, known as clay lump, achieved considerable popularity in East Anglia during the nineteenth century (McCann, 1983; Keefe, 2005). A wide variety of soils are suitable for adobe construction. Sufficient fines are required to develop cohesion and large gravel particles should be removed. The advantages of adobe include pre-drying of materials before construction, reducing shrinkage compared to cob, and pre-manufacture allowing transport, over short distances, and quality testing (often by simply dropping onto the ground or through handling). Where shrinkage is high, due to higher clay content or clay type, the addition of 1–3% straw by mass (typically) is a widely used form of soil improvement. Straw also increases robustness of blocks as well as reducing weight. In response to soil properties, climate, culture and other influences, a

Figure 5 Traditional adobe manufacture, Mali

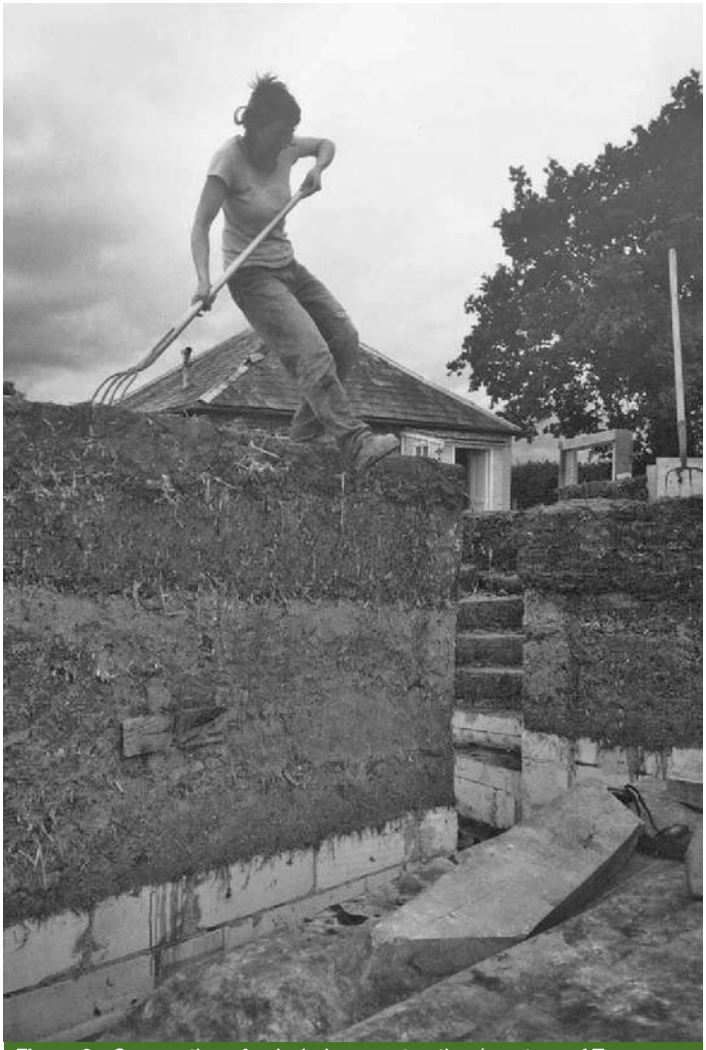

Figure 6 Compaction of cob during construction (courtesy of Tom Morton)

wide range of adobe sizes have developed around the world. Blocks should be easy to handle. Soils with limited clay content tend to use smaller, tile-size blocks. Cob blocks, available in the West Country, are a form of adobe block.

- Cob: cob is form of in-situ stacked mud construction. Soil and straw mixes are prepared to moisture content sufficient to allow moulding but also allow, safely, around 1 m of wall construction per day. Around 1–3% straw (by dry mass) is added to control shrinkage. The wet mix is stacked in layers and compacted manually in lifts of around 600–1000 mm per day. In general, work is carried out without formwork (**Figure 6**). Once completed the freshly compacted wall is cut or trimmed back, to produce slightly more uniform and flat surfaces, using a shovel or adze. The cut material is normally recycled. After around 1 week, depending on weather conditions, the second layer is added and in this way the wall is built up. Cob is arguably more suited to the UK climate as it is more tolerant to inclement weather during construction than rammed earth. After construction and drying, most cob walls are lime rendered or plastered. Walls are typically 400–800 mm thick. Openings are formed during construction. The large wall thickness and slow rate of construction limits scope for wider uptake of cob in modern construction.
- Extruded clay: a comparatively modern technique for production of unfired clay elements (including bricks, blocks and panels) adopted from the fired clay manufacturing industry. Processed (blended, crushed, screened, treated) clays are mixed with water to consistency to enable plastic extrusion through a die. Using the technique, combined with use of fibres, allows production of thin-walled blocks and panels. Industrial extrusion densifies materials, improving strength of materials. Production processes and materials used for fired clay can be readily adopted for unfired clay usage with little modification.
- Straw-clay: a relatively modern adaption of two traditional earth building materials. The mix proportions of straw to clay can be varied greatly to provide a composite material with density between 250 and 1200 kg/m^3. The clay content binds together the straw. Straw clay is non-load-bearing, used in conjunction with a timber or another structural framework. The uptake of straw-clay, despite early research (Morgan and Scott, 2002), has been limited, mainly due to problems of drying in the UK climate. Prolonged drying periods can lead to premature decay of the straw and/or persistently damp walls. Following construction, straw clay walls are plastered or rendered for protection.
- Earth floors: use of unfired clay in buildings is not limited to wall construction. Remarkably durable floors of earth can be formed using a variety of techniques, including rammed earth, poured earth (cob) and adobe blocks. To enhance water and abrasion resistance of modern floors, materials are commonly stabilised with additives, including linseed oils and waxes. Traditionally, floors were stabilised using cow manure, animal blood and urine. Floors need to be dried before applying any treatments likely to lock in moisture (linseed oil), otherwise mould growth can occur. Care and maintenance of floors is not unlike timber flooring, using regular reapplications of oils or waxes to build up protection of the floor.
- Plaster and renders: clay plasters are commonly used on internal surfaces of traditional earth walls. Through appropriate selection of materials and application, clay plasters provide hard-wearing surfaces. Modern pre-bagged plasters, available from companies such as Claytec and Terrafino, are primarily silt and fine sand, with limited clay content. The application of clay plasters requires a different technique to gypsum plasters, with less re-working and acceptance of slightly rougher surface finish. If over-worked with floating tool-clay plasters can dust and lose integrity with substrate surface. The limited training in use of clay plasters is currently a severe limitation to the wider uptake of these materials in modern construction. The use of external renders on clay walls is less common in the UK, but widespread in counties such as Mali, where rain-damaged plasters are repaired annually. The addition of straw and other natural ingredients improves durability of renders.

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Material characteristics and specifications

Relevant material properties of interest in specification include density, strength, moisture movement, durability and hygro-thermal performance.

The density of earth varies depending on technique and method of compaction, but is typically between 1800 and 2300 kg/m^3. This decreases with the addition of lightweight additives, most commonly natural fibres, such that the density of cob is typically around 1600–2000 kg/m^3, whilst lightweight straw clay has a much higher proportion of plant fibre and a density in the range 250–1200 kg/m^3.

Compressive strength is measured by testing cubes or cylinders of material especially prepared for that purpose. For quality control purposes it is essential that material is representative of as built materials. Methods of preparation, drying and testing need to be carefully controlled and agreed. In natural materials, without additives such as cement, compressive resistance is developed through clay fraction and pore suction. This strength is therefore dependent on the material's moisture content. Most importantly strength is lost as material gets wetter, so earth must be kept sufficiently dry throughout its service life. The relationship between strength and moisture content for an extruded green clay brick is shown in **Figure 7**. Values for compressive strength are typically quoted for air-dry conditions. Typical (unconfined) values are given in **Table 1**.

Like other massive materials, such as masonry, the tensile strength of earth is often ignored in structural design.

Material shrinkage can be a significant factor in design and construction, especially for modern applications. Traditionally, cracks opening in traditional cob or adobe walls would be filled by daubing in some further material or covering with plaster. To ensure air-tightness and a high-quality finish material shrinkage needs to be understood. Drying shrinkage is limited by selecting materials containing low-plasticity clays and adding as little water as possible during preparation. Prefabrication of elements, such as adobe, allows blocks to shrink prior to use. However, there will still be shrinkage of the mortar and plasters once the wall is built. In in-situ techniques all shrinkage must be accommodated to ensure adequate levels of air-tightness. Using straw fibres is a further way to limit shrinkage and more evenly distribute cracking. Levels of shrinkage vary considerably, depending on soil properties, initial water contents and drying conditions. Shrinkage levels in excess of 1% (10 mm per metre height or length) can occur, however shrinkage is often much less. The period for full shrinkage to develop will also depend on the mass of wall and drying conditions, but massive damp clay walls can take many months to completely dry out.

Material	UCS: Nmm^2
Rammed earth (natural)	1–5
Rammed chalk	0.4–1.5
Cob/adobe	0.5–1.5
Extruded clay brick/block	1.5–4
Compressed earth block	2–10

Table 1 Unconfigured compressive strength

Durability of earth walls is primarily concerned with its resistance to water, generally in the form of rainfall. However, for some exposed walls, such as rammed earth or plasters, resistance to dusting (abrasion) is an important characteristic. For many earth walls the problem of weathering is overcome by limiting use internally or protecting with weather-proof details (roof overhangs) and renders. For rammed earth, damage can be more likely to occur during construction through rainfall. However, through appropriate selection of materials it is possible to leave walls exposed to driving rainfall without undue concern (**Figure 8**). Gravel content is exposed, following washing

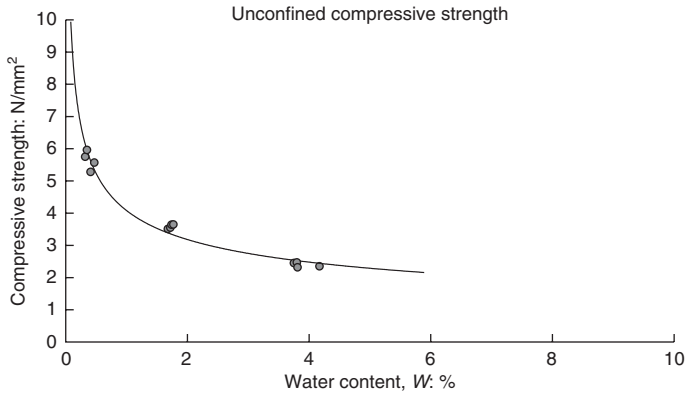

Figure 7 Relationship between compressive strength and moisture content for unfired clay brick (adapted from Lawrence *et al*. 2008)

Figure 8 Modern rammed earth wall, Austria

out of surface fines, to leave weather-resistant material. Rapid pre-testing of materials is not straight forward. Although there are some spray erosion tests, their correlation to field performance is limited and inconsistent (Thomson *et al.*, 2008). Precedence can be the most reliable means of material appraisal. Water resistance can be improved by addition of materials to the mix or the use of surface treatments, including linseed oil, waxes and chemical sprays (sodium silicates), as well as traditional uses, such as cow dung and blood.

Earthen materials are hygro-thermic. Clay minerals respond to environmental conditions, releasing water vapour to dry environments and absorbing humidity from damp (humid) environments. This property can be used favourably in buildings to passively regulate internal conditions. Monitoring has shown that ambient relative humidity (RH) in clay buildings in northern Europe can be limited to between 40–60%, only fluctuating with daily use in bathrooms and kitchens. Hygro-thermal performance also increases a material's thermal resistance. This aspect of clay materials requires further research and development, but has potential benefits for health (by limiting condensation) and storage of sensitive archival material (Padfield, 1998).

Modern applications and the future of earth building

The renaissance in traditional methods of construction, including cob, adobe and rammed earth, over the past 10–20 years has helped raise the profile of earth construction in modern construction, as well as within the conservation sector. A number of new projects have been completed successfully using traditional techniques, including domestic housing, public buildings, office blocks, educational buildings and exhibition spaces. During this period a number of earth building guides have also appeared (Walker *et al.*, 2005; Morton, 2008; Keefe, 2005), although there is, as yet, no national standard.

Modern examples of cob building, that largely follow traditional practice, include a number of houses, a series of small buildings at the Eden project, and the Genesis Project Earth Pavilion at the Somerset College of Art & Technology (SCAT). There has been little change or innovation in cob construction in response to modern building demands.

Cob (adobe) blocks are available from a small number of suppliers. To date, these have been used primarily for the repair of historic cob buildings, although a small number of new projects (Eden; SCAT) have also been completed. Cob blocks have potential to be more readily adapted to modern construction, as they resemble solid concrete blocks.

There has been a number of award-winning and high-profile rammed earth and rammed chalk projects over the past decade, including The Kindersley Centre, The Pines Calyx building, The Eden Project Visitors' Centre, The Centre for Alternative Technology, and the Rivergreen Centre (**Figure 9**). Success in these projects has stemmed from successful cooperation of the design and construction team combined with a supportive client. Although there have been innovations in formwork and compaction systems for rammed earth, there have been little further changes to traditional materials and methods. However, cement stabilisation of rammed earth has enabled a layer of rigid foam insulation to be buried into walls to improve thermal insulation whilst maintaining external finish.

Figure 9 Rammed earth, Rivergreen Centre, Durham

The large solid wall thickness, high cost of construction, poor thermal insulation and limited skilled workforce are some of the restrictions that will prevent traditional earth building techniques from making a major impact on the wider construction sector. However, there is scope for earth building to make a much larger impact, but to do this it must address concerns about wall thickness, cost of construction, durability, thermal performance and skills. Modern extruded unfired clay masonry offers the potential to develop unfired clay materials suitable for demands of modern construction (Morton, 2008). There are currently a variety of products available, including Ibstock's Ecoterre brick and Nuterra block from Akristos. To deliver thin clay masonry walls also relies on development of compatible mortars (Lawrence *et al.*, 2008). Clay plasters are also able to play a significant role in modern construction, but issues of skills training and supply of materials have yet to be addressed. As well as structural robustness, reducing wall thickness also has implications for resistance of walls to water damage.

Earth building has many attractive qualities, including low embodied energy, low toxicity, high aesthetic appeal and passive humidity regulation. Earth has the capacity to deliver high-quality, low-carbon buildings. There have

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been recent innovations in clay building that have opened up the opportunity for wider uptake. Future innovations are likely to include prefabricated panels and novel composite wall systems.

Building with lime

Background

Prior to the twentieth century, lime was the primary binder for many construction materials, including masonry mortars, plasters, renders and weak concretes. The production and use of lime probably started in the UK with the Roman occupation, although the use of lime elsewhere is much older (Holmes and Wingate, 1997). Prior to industrialisation, lime was produced in small local kilns and often prepared on site. The quality of material produced varied. In the nineteenth century Portland cement eventually replaced lime, initially as a binder for concretes, with obvious structural benefits, but subsequently in the twentieth century also as the preferred binder for mortars and finishes.

Portland cement is a much stronger hydraulic binder than lime. This has particular benefits for structural concrete, but the development of higher early strength, combined with more consistent performance also enabled cement to replace lime in mortars. Higher early strength allows faster rates of wall construction and scope for thinner wall sections. Although stronger cement mortars enabled faster construction than historically possible, their use has seen the introduction and widespread use of movement joints in masonry walls. Softer, weaker, lime mortars historically accommodated movements in masonry, self-healing small cracks that may develop. The greater capillarity and vapour permeability of lime mortars prolongs the life of masonry units, as moisture passes through the mortar joints rather than through the masonry units. Consequently, any frost damage and salt crystallisation is largely limited to the mortar joints, which are much more easily repaired (by repointing). The lower strength of lime mortars enables greater recycling and reuse of historic bricks and blocks.

Production of lime

Most limes are formed from calcium carbonate materials, including limestone, chalk, shells and coral. In the UK the raw material is primarily limestone. Crushed and processed calcium carbonate is burnt (calcined) in a kiln at around 900°C, producing calcium oxide (CaO) as well as carbon dioxide. The calcium oxide is turned into calcium hydroxide ($Ca(OH)_2$) by adding water, in a process known as slaking. Calcium hydroxide, in the form of dry powder or putty, is the product used by builders, mixed together with aggregates (sands, crushed limestones), to produce mortars, plasters and renders. On exposure to atmospheric carbon dioxide the $Ca(OH)_2$ converts back into calcium carbonate in a slower process known as carbonation. Carbonation commences from the exposed external surfaces into the section depth. The rate of carbonation varies depending on environmental conditions and material density, slowing with time as the porosity of material increases with carbonation. For example, once the material has dried sufficiently, initial carbonation rates in air lime mortars have been measured at around 0.10–0.25 mm/day (Lawrence *et al.*, 2008). Carbonation can only occur where there is sufficient humidity (water vapour), as the atmospheric carbon dioxide is dissolved into water to form carbonic acid which reacts with the calcium hydroxide. The process of burning, slaking and carbonation is known as 'the lime cycle'.

The quality and characteristics of the lime produced is dependent on factors such as qualities and processing of the limestone material, kiln burning process and additives included during manufacture. Pure forms of calcium carbonate will produce pure forms of calcium hydroxide. On use these materials only harden by carbonation, the so-called non-hydraulic or air-limes. Impure calcium carbonate, containing clays or other forms of silica and alumina, produce hydraulic limes through the production of pozzolanic material during burning. Hydraulic limes set under water, by chemical reaction similar to cement pastes, hardening further by carbonation when exposed to the air. The hydraulic set of limes is much weaker than those developed by Portland cements, but sufficient for a variety of construction uses. The initial strength gain from the hydraulic set of limes is typically much faster than carbonation.

Historically it was common practice to deliver calcium oxide to the building site, to be slaked by the mason or builder. In some cases slaking was carried out as part of mixing for construction process (hot-lime concrete foundations for example) (Holmes and Wingate, 1997). However, due to important health and safety and quality control of production, slaking is now undertaken as part of the factory process in all but the smallest conservation projects. Slaked dry limes can be stored as dry powders or in the case of air-limes as lime putties, ready mixed with water into paste and stored under water to prevent premature carbonation. Puttys may be stored for many years before use.

The embodied carbon dioxide of hydraulic lime is around half of Portland cement; around 400 kg of CO_2/tonne compared to 800 kg of CO_2/tonne for Portland cement (Hammond and Jones, 2008). However, the widespread use of blended cements, incorporating industrial pozzolanic by-products, has significantly reduced embodied carbon dioxide contents of cement products, so the carbon benefits of using lime in-place of cement are less evident. However, through carbonation, lime will generally reabsorb a great deal of the original chemically bound CO_2 much quicker than does cement. Other environmental impacts of lime

(and cement) manufacture include raw material extraction during quarrying, transportation, dust production, and habitat damage. The use of lime, in place of cement, needs to be considered against other potential benefits. Increasingly cement is blended together with lime to produce high-performance lime-based binders, known as formulated limes (BS EN 459: 2001).

Construction uses

Lime is used as the binder together with aggregates to form mortars, plasters, renders and concretes. Through choice of lime binder, aggregate type and mix proportions it is possible to achieve materials with a variety of characteristics suitable for a range of construction uses.

Aggregates used with limes include silica sands (naturally occurring and sharp crushed sandstone), crushed calcium carbonate stones (limestone, chalk, shells), earth or crushed waste brick or clay tiles. Selection of aggregate has a very important influence on the properties of lime material formed. Research has shown that some calcium carbonate aggregates can produce significantly higher-strength material than silica sands (Lawrence, 2006). The use LECA (lightweight (density 300–500 kg/m^3) expanded clay aggregate) has grown in recent years, offering improved thermal insulation qualities for limecrete floors for example. LECA is available in a variety of sizes (gradings), replacing natural stone and sand aggregates. Although LECA has a higher embodied energy than natural aggregates, this is arguably offset by the improved thermal insulation qualities of the finished material.

Mix proportions for lime mortars, plaster, renders and concretes are typically 1:2 to 1:3 parts lime binder to aggregates (by volume). A mix ratio of 1:3 ensures for most mortars that sufficient binder is available to fill all voids between grains, coating all aggregates and optimising material use.

As lime materials develop strength at a much slower rate than cement- and gypsum-based materials, it has benefits and drawbacks. Lime mixes tend to remain workable for longer periods and accommodate subsequent movements, but will also slow possible rate of construction compared with stronger materials.

The introduction of dry-silo mortar production on sites has done much to enable greater use of hydraulic lime mortars in modern construction (**Figure 10**). The improved quality control and performance of materials provided by dry-silo production has improved confidence in the quality of lime mortars. Other innovations that have enabled greater use of lime materials have included: spray application of plasters/renders; production of blended formulated lime binders that combine benefits of lime with some of the advantages of cement (faster strength development; consistent performance). Other innovations have included new composite materials, including hemp-lime (see later).

Figure 10 Use of dry silo supplied materials on site have supported uptake of lime mortars (courtesy of Lime Technology Ltd)

Lime material performance

The properties of lime materials are dependent on many factors, including grade of lime used, mix proportions, initial water content, age, curing conditions, method of placement, and aggregate properties (Allen *et al.*, 2003). As a general comparison, the 91-day compressive strengths of air-lime and natural hydraulic lime (NHL3.5) mortars (1:2 binder:sand ratio by volume) are around 0.4–1.0 N/mm^2 and 1.0–2.0 N/mm^2 respectively. For design, more specific properties should be determined from the manufacturer, testing or, where available design standards.

Benefits of lime use

- Fewer movement joints in masonry: Traditional walls with lime walls were built for hundreds of metres in some cases without movement joints. Masonry codes for cement-based mortars specify movement joints at 6–12 m centres (BS 5628: 2005). However, the use of cement mortars has also led to walls with far greater slenderness ratios than traditionally built with limes. Although further work is required to establish the movement properties of lime-mortared masonry, where there is historic precedence, including some early uses of cavity walls, significantly fewer movement joints in lime-mortared masonry would seem appropriate (**Figure 11**).
- Limes have lower embodied energy and CO_2: This stems from lower energy use in calcining compared to cement production, but also greater and faster carbonation of lime. However, this advantage needs to be carefully balanced with how the binder is used; potential benefits of lime binder over cement can be easily lost if a far greater volume of lime is required in the mix.
- Beneficial lower strength of lime mortar allows greater re-use of masonry units on recycling and reduces the need for movement joints.
- Improved water vapour permeability and capillarity protects masonry units and other construction materials, prolonging the life of the building fabric from moisture-related decay.

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Figure 11 Oxford University Club used lime mortars (courtesy of Lime Technology Ltd)

Greater vapour permeability could improve the health of living spaces by allowing moisture generated within a building to readily escape, lowering the risk of condensation.

Despite these benefits, barriers to greater uptake of lime materials currently include: cost, rate of construction, concerns over consistency of performance, and limited design guidance.

Hemp-lime building

Background

Hemp-lime is a lightweight composite building material that combines renewable plant based aggregates (fast growing hemp shiv) with a lime-based binder. Hemp-lime is a non-structural material used to form solid walls (**Figure 12**), roof and under-floor insulation. It is mainly used together with a structural frame, typically of timber construction. The lime binds together the hemp aggregates, giving the material modest mechanical resistance, but also protecting the hemp shiv from biological decay, mainly through its high alkalinity, as well as providing fire resistance. Hemp-lime has been under development over the past 10–15 years. Much of this work has been undertaken in France, Belgium and more recently the UK (Arnaud and Samri, 2006; Bevan and Woolley, 2008; Evrard, 2006; Evrard *et al.*, 2006; Yates, 2002).

Materials

Hemp shiv

The shiv (bio-aggregate) used in hemp-lime is sourced from the stem of the hemp plant (*cannabis sativa*). Hemp is an industrial bast fibre crop, growing to around 4 m high in as many months (April–August in the UK). On harvesting, the plant stems are often, but not always, cut and left to partially rot in the field before further processing (a process known as retting). Thereafter the higher value outer fibres

Figure 12 Hemp-lime panel

are stripped away from the inner stem. The shiv is made by cutting the inner stems into chips between 4 and 25 mm in size. The shiv is comprised of around 37% cellulose, 17% hemi-cellulose, 22% lignin, 5% pectin, 9% extractives and 10% other elements (ash, silica, tannin) (Bevan and Woolley, 2008). Around 60% by mass of the hemp plant is comprised of shiv, with typical total plant yields in the UK of around 8–12 t per hectare.

Hemp shiv is very hydrophilic, with a capacity to absorb up to 4.5 times its own weight in water within its cellular structure. After just 60 s of immersion the shiv absorbs around 1.5 times its own mass in water. Rapid de-watering of the fresh lime by the hemp can significantly impair the hydraulic set of the binder. This has resulted in poor-quality materials when using just natural hydraulic lime binders. Grading of the hemp shiv is also important for high-quality construction. A great deal of fine dust can be produced during processing. For efficient use with a lime binder, dust levels should be minimised as it increases the rate of water absorption, thus impairing the hydraulic set of the lime. The hemp shiv should be in good condition, though it is likely to contain a variety of fungal spores and moulds from the fields.

Binder

Earlier uses of hemp-lime largely used natural hydraulic lime binders. The results of this early work were variable in quality. Although it is possible to achieve a satisfactory material with hydraulic limes, incomplete hydration (flouring) of the binder within the mix is commonly observed, leading to poor-quality friable materials. On some occasions persistent damp patches, leading to mould growth, have been observed as well. This has been caused by the surface set restricting drying. The slow setting times of hydraulic lime bound hemp is also a concern for modern construction works.

These problems have led to development, by companies such as Lhoist and St Astier, of hydraulic binders specifically for hemp-lime work. These binders are blends of hydrated lime (typically 50–60% by mass), Portland cement (typically 40–30% by mass), and 5–10% by mass of pozzolanic additions (such as ground granulated blast-furnace slag). In addition, other mix ingredients include admixtures to encourage formation of fine pores and capillaries within the set binder matrix, surfactants to disperse materials and ensure a uniform mix, and water repellents to control water absorption of hemp shiv. Formation of micro-capillaries is reported as important for successful use of hemp-lime as it ensures the material maintains the hygroscopic characteristics of the shiv. The lime-based binder initially sets hydraulically and subsequently more slowly by carbonation. Carbonation of lime and cement is a key aspect of material performance, as it reduces the overall carbon footprint of the material. The rate with which carbonation takes place and the extent to which carbonation occurs is the subject of current research.

Hemp-lime construction

Hemp-lime materials are either cast (lightly tamped) inside temporary formwork (**Figure 13**) or spray applied (**Figure 14**). The introduction of spray techniques provides an opportunity to innovate with materials and mix formulations. Mix proportions of hemp shiv, lime binder and water vary depending on use (wall, floor or roof mixes) and the method of application (casting or spraying). For cast wall applications, the recommended mix proportions are 16 kg of shiv, 48 kg of water and 36 kg of lime binder to achieve around 0.2 m^3 of material. For spraying, the hemp and binder proportions are the same, but the water content is reduced. Sufficient lime is required to bind together the shiv matrix. A sufficient balance of water is required to allow the lime binder to set properly whilst balancing the hydrophilic action of the hemp. Cast walls are lightly tamped horizontally inside temporary formwork. Sprayed hemp-lime uses equipment originally developed for spraying concrete, applied in vertical layers against one layer of permanent formwork. Following initial set and drying, the walls are finished off with a breathable plaster or render, generally lime based.

Figure 13 Cast hemp-lime (courtesy of Lime Technology Ltd)

Properties of hemp-lime

Mechanical properties

Although hemp-lime is a non-structural building material, it is important to understand the mechanical properties of the material as it characterises its integrity and development with age. In addition, hemp-lime external walls are required to resist lateral wind loads, so it is important to understand its flexural strength. Hemp-lime may be used to contribute to racking shear resistance of timber stud walls, so knowledge of shear resistance of hemp-lime is also necessary. The compressive strength and elastic modulus characteristics of hemp-lime mixes are measured using test cylinders prepared in the laboratory. Displacement measurements undertaken during load testing

Figure 14 Spraying hemp-lime (courtesy of Lime Technology Ltd)

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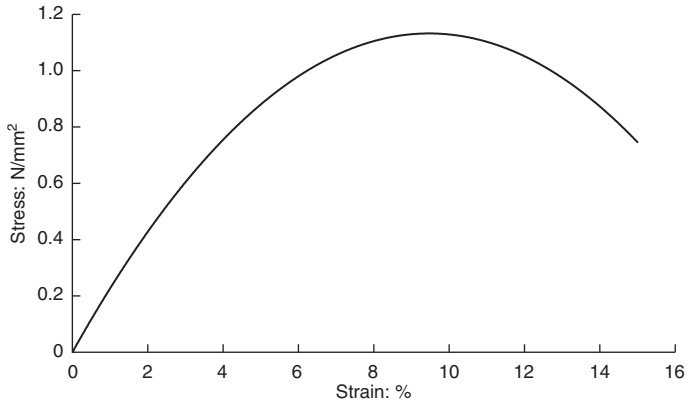

Figure 15 Stress–strain for hemp-lime material (Dry density = 450 kg/m^3)

establish stress–strain properties. A typical stress–strain curve for hemp-lime material is shown in **Figure 15**. Typical compressive strengths vary (depending on age, density and mix design) between 0.2 and 2 N/mm^2, with elastic moduli in the range 10–100 N/mm^2.

Water transport characteristics

Hemp-lime is a breathing wall material, allowing water vapour to pass through its structure. This helps to regulate internal building humidity, minimising risks of condensation and so improving the health of building space. The water transport characteristics are also related to the hygroscopic characteristics of the hemp. Breathing wall material is likely to improve durability of timber or metal frame structures embedded within it by preventing prolonged build-up of moisture. It is important to measure and understand the porosity, capillarity, permeability and water vapour permeability properties of hemp-lime materials. The drying times for in-situ hemp-lime walls are influenced by ambient weather conditions and potentially might influence construction finish times. Prolonged drying in cold and damp periods may delay occupation and, potentially more importantly, increase the risk of decay.

Thermal transmittance

Thermal transmittance (also known as the U-value (W/m^{2}K)) is the rate of heat transfer through one square metre of a structure divided by the difference in temperature across the structure. It is a function of the thermal conductivity properties of the constituent materials, measured under 'steady-state' conditions in a hot-box, and their thickness. It remains a common method of comparing thermal insulating qualities of different constructions. Values of thermal conductivity for hemp-lime have recently been established, varying with density, between 0.07 and 0.15 W/mK.

Hygroscopic properties

Hygroscopic materials absorb and release water vapour in response to changing atmospheric (temperature and humidity) conditions. Traditional hygroscopic building materials include timber and unfired clay. It is now increasingly recognised that this behaviour has beneficial health aspects by controlling internal humidity levels and preventing condensation, but also enhances a material's thermal performance. In response to increasing temperature, and changing relative humidity, latent energy is used to phase change absorbed water into vapour as it is released from the material. Conversely, as the material cools, heat energy is released and the water vapour is reabsorbed. Hemp-lime is a hygroscopic material. The hemp shiv absorbs and releases water with changing environmental conditions. The open pore structure of hemp-lime contributes to this characteristic. Some limited work on hygrothermal performance has largely been laboratory-scale studies and some limited computer simulations (Evrard *et al.*, 2006). This work has shown hemp-lime to have far superior thermal performance than simple thermal conductivity values would suggest.

Durability

As with other plant-based materials, such as timber and bamboo, maintaining low moisture levels in use, through good building practice and maintenance, prolongs the life of hemp. Durability of the hemp shiv is enhanced greatly by the protection against fungal and mould decay provided by the lime. The high alkalinity (initially around 12 pH) of the lime binder effectively protects the hemp against decay. As the lime carbonates, its alkalinity reduces, though experience to date suggest this is still generally sufficient to protect the hemp against fungal attack.

Other renewable materials in construction

Renewable materials, using plant and to a lesser extent animal-based products, has formed the basis of vernacular building for millennia. Common features of these materials are renewability, low compressive resistance and relatively poor durability. Single fibres, grasses, reeds or leaves are usually too weak to support their own weight, but bundled, twisted, compressed or woven together in groups used for a variety of structural and non-structural building applications. Applications include: soil reinforcement; ropes and twines; thatch roofs; reed boards and matting; woven leaf panels (roof, walls); reeds, bundled or tied together as boards for various uses as columns or beams. In addition to applications outlined above, common renewable-based construction materials include:

- Bamboo: widely used in Asia and Latin America for a variety of temporary and permanent construction applications. Uses include scaffolding, structural elements (beams, columns, and roof structures), external cladding and internal finishes (Design Museum Vitra, 2000; Jayanetti and Follet, 1998).

- Leaves: variety of plant leaves, sometimes woven, are used for roof, wall and floor coverings, mainly in tropical countries.
- Straw: traditionally widely used for thatching roofs, fibre addition to earthen materials and most recently for straw bale construction.
- Reeds: used for thatch and panels.
- Fibres: widespread use of plant-based fibres, such as sisal, flax and hemp, as reinforcement for composite materials as well as formation of quilted batts.
- Animal products: animal based additives include tallow, casein, blood and urine for improvement or treatment of other materials. Fleece (sheep's wool) provides an excellent insulation material.

This list does not include the most widely used renewable material: timber. As a well-established conventional material, timber is covered elsewhere in this work. This is not a comprehensive review of renewable materials in construction, but rather an insight into potential for using plant-based renewable products. Uses of renewable materials in the UK are primarily for insulation products, roofing and cladding, and natural fibre additives.

The advantages of using renewable plant materials include: potential to use locally available and often abundant material (such as straw); renewable (sustainable) resource that decreases demand on non-renewable resources; carbon sequestration during growth and storage until composting; ecological disposal (composting) on end use; healthier buildings, new markets for farming; novel hygroscopic properties. Potential problems using such materials include: poor durability if not protected from moisture or allowed to dry out; vulnerability to insects, rodents, fungi and rot; and a potential fire risk (e.g. thatch).

Insulation products

Due to their low density a variety of insulation products produced using natural fibres are now available, and now one of the most common uses of such materials in construction. The relatively high cost of these materials has limited uptake.

Hemp-fibre insulation

Hemp-based insulation, such as Isonat®, is made from industrially produced fibres. Products often include polyester fibres, to improve 'loft' and stability. Hemp-fibre insulation quilt is suitable for external walls in timber- and steel-framed buildings, as well as for internal applications including walls, suspended floors, lofts and roofs. It has a thermal conductivity of 0.039 W/m K and a density 35 kg/m^3. Like other natural materials it is hygroscopic, which enhances its heat storage capacity. Hemp insulation has a good resistance to pests (e.g. moths and rodents) and is treated with a boron salt compound to improve fire resistance.

Sheep's wool insulation

Sheep's wool has developed into a very effective natural insulating material. It is a wholly natural fibre, with good natural fire resistance. Wool is able to absorb up to 33% additional water within its fibre structure without formation of free water. It is also a hygrothermal material. Wool also has a natural loft and its natural helical crimp and bulky structure traps a lot of air between its fibres. Wool is, however, prone to attack by insects, especially moths, and therefore requires treatment. Wool degrades on prolonged exposure to sunlight. It is available in batts either 400 or 600 mm wide and between 50 and 100 mm thick and 1200 mm long. Wool has a density of 22.5 kg/m^3 and thermal conductivity of 0.037 W/m K. 'Thermofleece' is one of the few natural materials to have BBA certification.

Wood fibre insulation

Wood fibre insulation board, manufactured without glue or preservatives, can be used to insulate roof and wall constructions. Currently imported from Austria, boards are available in a variety of thicknesses, between 20 and 120 mm, and panel sizes. As well providing effective insulation, wood fibre boards also have good acoustic qualities and exhibit hygroscopic properties. However, wood fibre insulation is not water resistant, and so it must be protected throughout its life. Care is also required to avoid contact with solvents and fresh wood preservatives. The thermal conductivity of wood fibre boards is 0.040 Wm/K (at a density of 160 kg/m^3).

Thatching

Thatch is a flexible roof covering, able to cope with many different shapes. It provides a weather-tight, durable (up to 100 years) and aesthetically pleasing finish. Using renewable resources, thatch provides some thermal insulation (U value for 300 mm thick water reed roof at 45° is 0.35 W/m^2 K). Thatch reached particular popularity in rural Britain during the nineteenth century. Though thatch is a common and universal roofing material, presently it is not fashionable. Advantages include thermal properties, renewable resource, and its simplicity of construction. Its main drawbacks are limited durability, fire risk and high cost of labour.

Water reeds are the most durable thatching material, though cereal straw is also widely used. To drain water the roof pitch should be above 40–45°. The supporting structure should be sufficient to bear the weight of the thatch, which can be up to 40 kg/m^2 for reed thatch.

Durability of thatch roofs is dependent on ensuring even density throughout; loose patches tend to attract water, rotting more quickly and leading to leakage problems. Thatch is combustible and prone to fire damage. Chemical treatments are available, such as boron, to reduce fire risk. Fire risk is also reduced by positioning houses apart and

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good chimney design and on-going maintenance. Insect damage is also best dealt with through good design to minimise risk. The typical life span of long straw thatch is 20–25 years; combed wheat 30–40 years and water reed around 50 years.

Natural fibre composites

Composite materials are made from two or more materials mixed together. Materials are added together to develop properties (strength, stiffness, toughness, cost, shrinkage) not attained by any single component acting individually. Natural fibres, such as sisal and hemp or animal hair, are added to a variety of matrix materials, such as cement, lime and clay. Fibres can improve consistency when wet, inhibit shrinkage cracking and can enhance strength and ductility. Manufactured composites include straw-reinforced mud (blocks, cob), fibre-reinforced plaster, reinforced plastic and reinforced concrete. Building applications of natural fibre composites include: partition wall panels; roof panels; building blocks; architectural mouldings; insulating panels; and timber substitutes. Advantages of natural fibres include: lower density; renewable production; low toxicity; and, good insulating properties. Natural fibre composite materials are treated in greater depth elsewhere. Physical characteristics of composites are dependent on properties and compatibility of fibres and matrix as well as fibre quantity, length and orientation.

Following the health problems associated with asbestos fibre use, various plant-based alternatives have been investigated as replacements (Savastano, 2008). Some untreated plant fibres are not suitable in a cement matrix, such as bagasse fibres, as its high sugar content retards the setting of the cement paste. Sisal and coir have been widely investigated. Fibre contents vary between 1 and 2% by weight. Fibre-reinforced cement materials are used to make variety of products, including corrugated roof panels. Plant-based fibres in cement can deteriorate in an alkaline environment resulting in brittle behaviour. This can be overcome by reducing the alkalinity of the cement, using a high-alumina cement or replacing 50% cement with pozzolan (silica fume), or protecting fibres by coating or sealing, or pressure and steam curing to accelerate carbonation. Durability concerns can arise from high water absorption (moisture movement of fibres), low fibre stiffness, and susceptibility to fungal and insect attack.

Prefabricated straw bale construction

Straw bale construction has been used, to varying extents, for over 100 years (King, 2007). Bales may be used to form modest load-bearing walls or as in-fill material in framed construction. Bales are a cheap and abundant renewable crop-based resource. However, despite success in a small number of projects, wider uptake has to date been very limited. During construction straw bales require protection from inclement weather conditions. Although modular sized (nominally $1000 \times 450 \times 350$ mm), bale dimensions (length in particular) and compaction density can vary significantly. This causes problems for construction and detailing. Load-bearing construction relies on a period of settlement prior to the application of the render, prolonging the period of construction. Barriers to broader commercial uptake therefore include uncertainty over technical performance characteristics (especially durability and fire resistance of straw) and high manufacturing costs of on-site construction subject to delays due to inclement weather.

The development of prefabricated straw bales seeks to overcome these barriers through the off-site production of high-quality panels in-filled with straw. One approach uses timber-framed panels in-filled with straw bales (Modcell, 2008). Panels are typically 490 mm thick, with varying heights and widths to suit. The thermal transmittance of these panels is around $0.13\,W/m^2\,K$, nearly three times better than that required by the building regulations. In construction, the straw bales are compressed vertically to improve their structural robustness and dimensional regularity in preparation to receive plasters, renders or timber-based fascias. On completion of straw infill and compaction, the internal and external faces of the panels are covered in a formulated lime-based render (**Figure 16**). In an alternative 'dry' method the panels can be covered in timber-based fascias. It is important that the bales remain for prolonged periods at moisture content levels below which biological decay is likely to occur (around 25%). The facings must therefore protect the bales from direct weathering but also allow water vapour to escape as part of a breathing wall system.

Carbon benefits of renewable materials

Present specifications require that $1\,m^3$ of hemp-lime wall mix contains 110 kg hemp shiv and 220 kg formulated lime binder. The shiv, based on typical material density and constituent lignin and cellulose contents, has a locked-in CO_2 content of 1.84 kg/kg of shiv. The lime binder has CO_2 emissions of 0.43 kg/kg of binder (assuming full carbonation of the lime). The total negative embodied carbon is therefore 110 kg of CO_2/m^3 or 33 kg of CO_2/m^2 for a 300 mm thick wall. However, the density of the hemp can vary and more significantly the density of the finished wall can also vary according to the method of placement.

Environmental benefits of straw bale construction are largely realised through the use of straw. In its growth, straw sequestrates atmospheric carbon dioxide, storing the equivalent of 0.42 kg of CO_2 per kg of straw. Throughout the life of the building, this carbon remains locked up in the fabric of the building. Following harvesting there is little further processing, so straw

Figure 16 Lime rendering prefabricated straw bale panel

maintains its low embodied energy. The low carbon benefits of straw are reduced by use of protective lime render, but compared with other finishes this is still the lowest impact solution. The timber framework uses renewable, plantation grown, European softwood, and a further store of CO_2. The net embodied carbon of a prefabricated straw bale panel is estimated to be around 70 kg of CO_2/m^2.

Using straw or hemp reduces demand on competing insulation materials, such as fibre-glass and expanded foam products, sourced from non-renewable high-energy materials. However, the benefits of renewable materials are not only embodied within the fabric but also realised through the significantly improved environmental performance of the buildings, derived from the very high values of insulation (U value = 0.13 W/mk for a standard straw bale panel) and excellent airtightness.

Future potential

Non-conventional, low-carbon, natural materials offer exciting new technologies, and broaden the range of materials available to the designer, builder and client. Despite their many benefits, their use in modern construction to date remains comparatively limited. In the past 10–15 years, the market, which had almost no activity, for materials such as unfired clay and straw bale has grown. Although it still remains a small fraction of mainstream construction, interest in these technologies has never been greater. Initiatives in response to climate change, including the Code for Sustainable Homes (Code for Sustainable Homes, 2006), has provided the framework and stimulus for this increased interest and encouraged growth in the use of lower carbon building materials. However, for significant mainstream uptake, important barriers to use remain to be addressed. These include higher initial construction cost, demonstration of reliable long-term performance, reduced reliance on traditional in-situ methods of construction, reliable supply chain for product delivery (including a skill base), and, development of supportive design guidance and standards. To enable this market development requires further research and innovation, supported by both government and private investment. Exciting future developments are likely to include nanotechnological studies and development of traditional and novel natural building materials. Although many low-carbon techniques are more expensive to build than competing materials and methods, this differential might reasonably be expected to reduce through future innovation and greater market share. The cost of not significantly reducing the environmental impact of our buildings will certainly be much higher.

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Chapter 73

Straw bale construction – state of the art

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Straw bale construction is a building technique by which the exterior walls of a structure are constructed of plastered straw bales, which in some cases act as the primary structural elements of the building. Although more research is required, straw bale construction has been proven both anecdotally and through numerous experiments to be an excellent alternative to typical stud-frame construction.

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Introduction

With the challenges of global warming and the increased cost of energy, homeowners, builders, architects and engineers are increasingly turning to an old technology made new: straw bale construction. A typical straw bale wall is illustrated schematically in **Figure 1**. Straw bales, which usually measure 350 mm by 500 mm by 800 mm, are stacked to form walls. Previous practice in North America has been to cover the bales with a wire mesh to enhance the bond with the plaster but, in many cases today, the conventional practice is to avoid the use of mesh. The bales are typically topped with a timber box beam. For post-and-beam construction, a timber frame provides the structural support, while straw bales provide an insulating infill. For load-bearing construction, the plastered straw bale walls provide all the structural support. The straw bales provide excellent, environmentally friendly insulation (as will be discussed in a following section), while the plaster applied to the bales provides strength and protects the bales.

Straw bale construction has many benefits:

1. Straw bale walls have very good insulation properties which are achieved without the use of commercially manufactured materials.
2. Straw bales are annually renewable and widely available in most parts of the world. They are produced locally which saves on transportation costs, lowering the embodied energy of the building dramatically.
3. The use of straw bales as a construction material discourages the disposal by burning of straw, a common practice in the past.
4. Straw bale construction can be a very inexpensive construction method due to its ease of construction and suitability for use of volunteer labour.

There are several drawbacks to current straw bale construction. The first is that it can be difficult to find a builder familiar with this construction technique. The production of straw bales has been optimised for the agricultural industry, rather than the construction industry. The width of typical bales available means that the walls of a straw bale house are thicker than with conventional timber or masonry construction. The properties of straw bales (especially width and density) can be highly variable, which can make it difficult to plumb walls and construct square corners. Straw bales are highly susceptible to rot due to moisture, so bales must be carefully protected until plastered. Finally, there is a lack of quantitative, scientific data regarding the structural performance and durability of straw bale construction.

The first documented use of straw bale construction was by settlers in the Nebraska area where there was a lack of timber for construction (Magwood *et al.*, 2005). With the invention of an automatic horse-powered baler, the settlers stacked bales of compressed straw to form walls, covered the bales with plaster, and topped them with a roof. Nebraska's oldest existing bale building, the Burke homestead, was constructed in 1903 outside Alliance, Nebraska. Although abandoned in 1956, the Burke homestead is still a sound structure that continues to successfully withstand Nebraska's wide temperature changes and strong winds (King, 2006).

In the 1980s straw bale was rediscovered as a method of construction by people looking to get away from the expense and environmental impact of conventional housing. By the early 1990s, a movement of straw bale building spread from the American south-west worldwide, and the number of structures built increases each year (King, 2006).

Most straw bale buildings currently being constructed are for residential applications, with single-storey buildings,

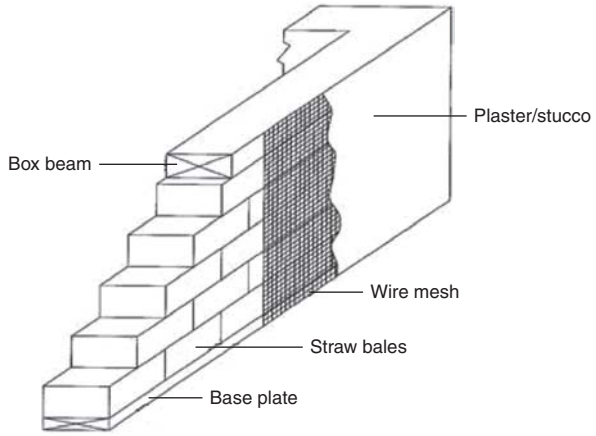

Figure 1 Typical straw bale wall details

as shown in **Figure 2(a)**, predominating. Some two-storey straw bale buildings have been successfully constructed, however (**Figure 2(b)**). Many of these projects have a number of other innovative green features, including solar panels, engineering wetland wastewater treatment and hydronic radiant floor heating.

There are no reliable figures available on the total number of straw bale buildings around the world. In 1996 it was estimated that there were at least 1000 to 2000 worldwide (King, 1996), and although the number is increasing year by year it is likely to have grown considerably since then. Several hundred are known to exist in the United States, mainly in the south-west. In Canada, the Ontario Straw Bale Building Coalition (OSBBC) maintains a database of straw bale buildings in Canada. As of 2007, approximately 100 projects were listed in the database, although not all projects are documented. The first house listed was completed in 1997. Over 600 straw bale buildings have been constructed in China between 1998 and 2003, and as many as 50 straw bale homes and 12 straw bale health clinics constructed in Mongolia (Kennedy, 2004). Thirty-one straw bale buildings are reported in Germany, and at least two each in the Czech Republic and Italy (King, 2006). There have been numerous straw bale projects in Norway, Denmark, the United Kingdom, South Africa, Australia and New Zealand, but again no reliable data on exact numbers constructed are available. Notably, there has been little straw bale construction in Central and South America due to moisture concerns (King, 2006).

Construction details

General

There are two main methods of assembling a wall using straw bales: (1) load-bearing, in which the bale wall carries vertical and horizontal load; and (2) non-load-bearing or post-and-beam, in which bales are used as an infill between or around a timber structural frame. **Figure 3(a)** shows a load-bearing straw bale building before plaster is applied, and **Figure 3(b)** shows a post-and-beam design.

For load-bearing construction, top and bottom timber plates are commonly cut to the width of the bale plus 25 mm on each side to accommodate a coat of plaster. Loose straw fills in the cavity of the bottom plate to provide a level surface for the bales to rest on. The bales forming walls are stacked in a running bond.

Bales

There is an important distinction between hay and straw. Straw is the cellulosic stem of a plant. Hay includes both stem and grain. The high carbohydrate content of hay

(a)

(b)

Figure 2 Examples of straw bale construction: (a) single storey; (b) two storey

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Figure 3 Straw bale wall before application of plaster: (a) load-bearing wall; (b) post-and-beam design

makes it much more susceptible to rot over time. The most prevalent source of straw is as a by-product of cereal grain production. A variety of grain types have been successfully used in straw bale construction. In the southern US, rice bales have found wide use. In many other parts of the world, wheat bales have been used predominately. In the past five years, hemp straw bales have been used successfully in several Canadian projects.

Straw bales typically measure 350 mm by 500 mm by 800 mm. There are two types of bale orientation: flat (the fibres of the bale oriented horizontally) and on edge (the fibres of the bale oriented vertically). When the bales are stacked flat, better insulation qualities are achieved while on-edge orientation results in larger interior square footage (for the same footprint) and allows for fewer bales to be used. **Figure 4(a)** shows bales stacked on-edge, and **Figure 4(b)** shows bales stacked flat. Note that in **Figure 4(b)** the bales have been dipped in a clay slip that is purported to improve the bond to plaster. This practice is becoming more popular in North America but is not common in

Figure 4 Orientation of stacked bales: (a) on-edge; (b) flat

Figure 5 Precompression of straw bale wall: (a) wrapping nylon strap around wall; (b) tensioning strap; (c) detail of strap connection

the UK where damp climate conditions complicate the curing process of clay-dipped bales.

Precompression of a bale wall is necessary in load-bearing construction in order to avoid settling of the wall after the roof loads (and in some cases second-floor loads) are applied. There is no standard method for doing this and a wide variety of methods are used in practice. **Figure 5** shows a popular in North America method that employs a nylon strap wrapped around the wall and fed through holes predrilled in the top and bottom plates. The strap is then tightened until the top plate of the wall is level. Typically, for a wall 2.4 m in height, the wall may be compressed approximately 100 mm.

Reinforcement

It is common practice to fix a mesh to the surface of the bales before plastering. In seismically active zones, a steel mesh is used to improve the ductility of the wall under earthquake loading. Even without seismic concerns, a mesh may be used as it is purported to aid in the bonding of the plaster to the wall assembly. **Figure 6** shows a polypropylene mesh commonly used in Canada and the US.

Figure 6 Affixing mesh to bale surface: (a) affixing mesh to top plate; (b) stitching mesh to bale surface; (c) detail of mesh

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Figure 7 Application of plaster to straw bale assembly: (a) trowel application; (b) hand application

The mesh is affixed to the top and bottom plates, and then stitched to the bale surfaces. There are no standards on the use of mesh, and many builders avoid using mesh, and simply apply plaster directly to the bales.

Plaster

Four basic types of plaster are used for straw bale construction:

- earthen clay plasters
- lime plasters
- cement/lime plasters
- gypsum plasters.

Most of the straw bale houses built in Canada and the US in the 1980s and 1990s used a cement/lime-based plaster. These plasters are strong and durable, but many builders in Europe and North America are now favouring lime or earthen clay plasters due to environmental concerns. The plaster is typically applied in three coats, using a trowel or by hand, as shown in **Figure 7**, or by spraying on the plaster. Magwood *et al.* (2005) provide more details on typical plaster mix designs for straw bale construction.

The methods of plaster application result in large variations of plaster thickness over the wall assembly. **Figure 8** shows the variation in plaster thickness for a 2.4 m × 2.4 m straw bale wall constructed at Queen's University, Canada using a cement-lime plaster and typical straw bale construction techniques, including trowel application of the plaster. The plaster thickness was measured by drilling into the plaster. The nominal plaster thickness was 25 mm, but the results show that the plaster thickness ranged between 20 mm and 60 mm.

Variations in bale surface and methods of render and plaster application can result in large variations of plaster thickness over the wall assembly.

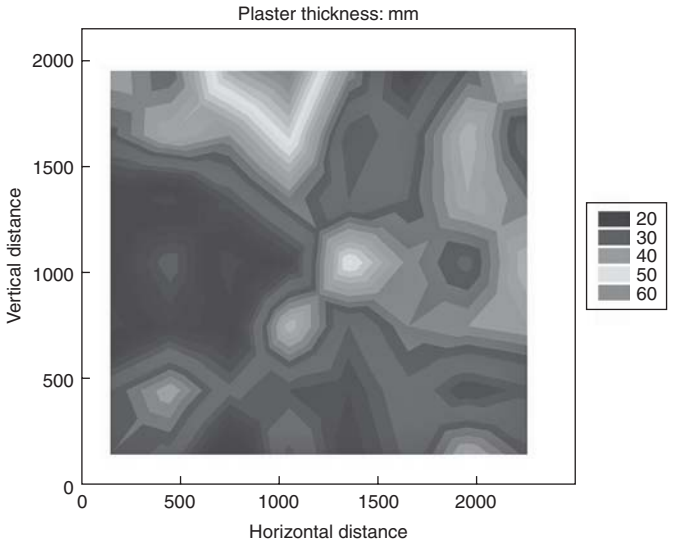

Figure 8 Variation in plaster thickness for typical straw bale wall

Figure 9 Haliburton 4C's food bank and thrift store (courtesy of Chris Magwood)

Significant variations in plaster thickness will influence cost of construction, resistance to fire and water borne decay, and final weight of wall construction. Variations in plaster thickness can be reduced by cutting the bale face, using a band saw for example, prior to construction. Where variable thickness is expected, it is important than any minimum specified thickness is maintained.

Performance of straw bale assemblies

In many respects, the knowledge regarding the performance of straw bale assemblies has increased enormously since its revival in the 1980s. There are a large number of projects in a wide variety of climates that are evidence of straw bale's excellent performance, showing that in many respects it meets or exceeds that of conventional residential construction. Testing and research has been conducted to address structural, durability, and fire concerns, and to quantify the thermal performance of straw bale construction.

Despite this, there remain many crucial gaps in the current literature on straw bale construction. First, much of the data on straw bale construction are available as reports or papers that have not been subjected to peer review in credible journals. Although this does not discount the findings, it does mean that straw bale lacks the credibility associated with well researched materials such as steel and structural concrete.

Second, there are many innovations being proposed and attempted in the field, but the research to support these innovations has lagged since it is still seen by many academics as a fringe area of research. These gaps need to be closed.

The following section will outline the current knowledge regarding structural performance, thermal performance, durability, and fire performance of straw bale construction.

Structural performance

Numerous experiments have been conducted in recent years on plasters, unplastered straw bales, plastered straw bales, unplastered straw bale walls and plastered straw bale walls. Unfortunately, due to a current lack of standardisation for straw bale testing methods, and the inherent variability of straw bale construction, the results reported in the literature vary significantly from experiment to experiment. Despite this, the results do indicate that with the appropriate design considerations, a plastered straw bale wall can be constructed to meet a wide range of structural requirements.

Plaster structural performance

Most straw bale structures in North America are plastered with a mixture of 1 part by volume Portland cement, 1 part by volume hydraulic lime, and 6 parts by volume sand. More recently, builders have experimented with earthen plasters in Canada and the US, including locations previously thought to be too damp. Earth-based plasters are made with soils that have a naturally occurring balance of sand and clay (Magwood *et al.*, 2005). They have significantly lower values of embodied energy as compared to those of lime/cement plasters. Generally, earthen plasters have much lower strengths and modulii of elasticity than lime/cement plasters.

Lerner and Donahue (2003), Ash and Aschheim (2003), Grandsaert (1999), Nichols and Rapp (2001), Faine and

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Zhang (2002), and Taylor *et al.* (2006) all tested various earth and lime/cement plasters typically used in North America for straw bale construction and found compressive strengths (at 28 days) ranging from 0.23 MPa to 27.6 MPa, with earth plasters having strengths at the lower end of this range. Lerner and Donahue (2003) also found cement-based plasters had modulus of rupture values of approximately 1.4 MPa, while Taylor *et al.* (2006) found modulii of elasticity values for a variety of earthen and cement/lime plasters between 400 MPa to 2500 MPa.

Taylor *et al.* (2006) investigated the effects of moisture content, drying time, drying conditions and clay content on the strength of earthen plasters. The earthen plasters were also compared to a plaster mixed using commercially available clay, and to cement-lime plasters. The results of this study show that earthen plasters can be used as a low-strength structural material. The plaster was mixed using soil consisting of 69% silt, 27% clay and 4% sand. The cubes and cylinders of cured plaster were loaded until failure and showed similar strength values to those of low-strength cement/lime plaster.

Increased clay content significantly increased the strength of the earthen plaster. For the earthen plasters, strength was found to increase linearly with clay content. Varying the moisture content at the time of testing by placing some specimens in a drying oven at 110°C and some in a moisture room with 100% relative humidity was intended to simulate plaster subjected to hot, dry weather or a heavy rainfall, with the laboratory air acting as the control environment. Placing the plaster in a 110°C oven substantially reduced the moisture content and increased the strength. Leaving the plaster in a humid environment increased the moisture content and resulted in lower strength as opposed to a lab environment.

In the UK, natural hydraulic lime plasters are commonly used for straw bale construction. The typical strength of these plasters (Maurenbrecher *et al.*, 2005) ranges from 0.32 MPa to 1.4 MPa (at 28 days) to 0.42 MPa to 1.9 MPa (at 240 days).

Individual bale structural performance

Bou-Ali (1993) and Watts *et al.* (1995) performed experiments on individual straw bales to find the load deflection behaviour of unplastered straw bales. Results suggest that the modulus of elasticity varies significantly from bale-to-bale, and that the density of the bales has a greater effect on the strength and stiffness of the bales than the type of bale.

Vardy and MacDougall (2006) conducted experiments on plastered straw bales and found compressive strengths ranging from 7.7 kN/m to 91.5 kN/m. In addition, the experiments conducted by Vardy and MacDougall (2006) studied the effects of the plaster thickness and strength on the overall strength of the plastered bale.

The bales used for experiments by Vardy and MacDougall (2006) had very consistent dimensions and plaster thicknesses which were achieved with the help of a wooden jig designed for this purpose. The jig was used to compress the bales to the desired dimensions and after the compression, as a guide to trim away the excess straw. A plastic frame was used around the edges of the bale to guide the plaster application and bring the plaster skins to consistent uniform thickness. This method reduces the variability of the experiments and ensures the clarity of conclusions.

Vardy and MacDougall (2006) tested plastered wheat bales oriented flat and on edge. The effects of the plaster thickness and strength on the overall strength of the plastered bale were examined. It was found that the strength of plastered bales oriented flat is 36% greater than those oriented on edge for a specific plaster thickness and strength. The plaster thickness was shown to have a greater effect on the plastered bale strength than the cube strength of the plaster. It was found that doubling the plaster thickness increased the average plastered bale strength by 65%, while doubling the plaster strength increased the average plastered bale strength by only 25%.

It is acknowledged that the uniform bale samples with consistent plaster thicknesses used for testing are quite different from actual construction. Even though in the real construction setting, the plaster and bale dimensions are highly variable, the results obtained in the study provide insight into the mechanics of straw bale construction and a solid base for further research. The conclusions can be used to guide design to obtain the optimum structural performance of the straw bale walls.

Bale wall structural performance

Bou-Ali (1993) conducted experiments on unplastered bale walls subjected to out-of-plane bending, finding the walls withstood between 1 kPa to 5 kPa of pressure. Grandsaert (1999), Faine and Zhang (2002), Zhang (2002), Fibrehouse Ltd and Scanada Consultants (1996) and Vardy and MacDougall (2007) all conducted experiments on the compressive strength of plastered straw bale walls, finding ultimate strengths of 20 kN/m to 90 kN/m for various tests conducted using a wide range of wall dimensions, plaster proportions and thicknesses, reinforcement schemes and bale types and sizes.

Vardy and MacDougall (2007) also noted the following key observations:

1. Assuming compression failure of the walls, the walls failed at 80% of the expected load.
2. Observations indicated buckling of the walls prior to failure with mid-height lateral deflections measured to be approximately 1.6 mm at failure.

3 A non-linear stress–strain concrete model was used to predict an approximate stress–strain response for the walls.

It is important to note that the experiments conducted by Vardy and MacDougall (2007) were on six walls built using jigs which provided very consistent bale and plaster dimensions. Again, it is acknowledged that, although in a construction setting the plaster application and bale dimensions have significant variability, the results and observations provide a means for understanding the structural behaviour and failure mechanisms associated with plastered straw bale construction.

In 2001, a number of significant in-plane cyclic experiments were conducted in a test programme funded by the California Department of Food and Agriculture and furthered the understanding of the seismic behaviour of plastered straw bale walls (King, 2006). Because of the small aspect ratio associated with plastered straw bale walls, they have shown an inherent stability ideal for resisting both minor and major earthquakes. It was shown that, with adequate detailing, a straw bale wall containing steel mesh reinforcement with a lime/cement plaster performs on a par with conventional plywood-sheathed stud walls, and can be considered to have a ductility R-factor of 5.5 as per the 1997 Uniform Building Code seismic design formula. An earthen rendered straw bale wall with plastic mesh has low ductility, but it was shown to have similar lateral-load resistance to that of a very lightly sheathed stud wall. It was also noted that even after significant lateral deformations, the walls still managed to sustain significant compressive loads, indicating an ability to avoid collapse even when subjected to the lateral loading indicative of a major earthquake.

Thermal performance

One of the primary benefits of straw bale walls is their purported excellent insulation properties. There is vast anecdotal evidence that the heating and cooling requirements for a straw bale home are significantly less than a conventional timber-framed home with glass-fibre insulation (Magwood *et al*., 2005). However, there is much less clear and defensible data in the literature quantifying the thermal performance of straw bale construction.

Among the thermal tests of straw bale construction include (King, 2006): (1) testing of bales using a hot-plate or thermal probe methodology; (2) testing of bale wall structures in a guarded hot-box facility; (3) monitoring of straw bale wall performance under ambient conditions (CMHC, 2002); and, (4) modelling of straw bale wall performance using known or assumed physical properties of the materials.

One of the most cited test results to date was a test in 1998 of a single plastered straw bale wall with bales 575 mm wide (King, 2006). An R-value of 33 (in customary US units of $ft^2 \cdot °F \cdot h/Btu$) was measured. This is equivalent to a U-factor of $0.172\,W/m^2 \cdot K$. On this basis, it is commonly reported that the R-value of plastered straw bale walls is 1.45 per 25 mm wall thickness.

A more recent study by Wimmer *et al*. (2000) measured the U-factor of plastered straw bale walls and reported $0.12\,W/m^2K$ for a wall with 350 mm wide bales. This is equivalent to an R-value of 47. The tests were conducted for Municipal Department 39 (Vienna), Austria, using ISO 8301.

Durability

Excessive moisture in a straw bale wall may lead to rotting of the straw. Thus, there has been much focus on understanding how to mitigate excessive moisture in straw bale walls. Straube (2002) summarises the current approach to mitigating excessive moisture in straw bale walls. The design of straw bale walls differs from that of conventional timber walls in that no vapour barrier is used as this would not allow bonding between the plaster and bales. Instead, plasters used for straw bale construction must be permeable enough to allow vapour diffusion and allow bales to dry out. Straube (2002) conducted an extensive study of the plasters and coatings used for straw bale construction, measured permeabilities, and made design suggestions for plasters on exterior and interior wall surfaces. It was noted that cement-based plasters have low vapour permeability. The addition of lime to a plaster increases its vapour permeability substantially. The vapour permeability of a plaster of 1 part Portland cement to 3 parts sand by volume was measured to be $1.7\,ng/Pa \cdot s \cdot m^2$. On the other hand, the vapour permeability of a plaster more typically used in North America, namely 1 part Portland cement to 1 part hydraulic lime to 6 parts sand by volume, had a vapour permeability of $10.3\,ng/Pa \cdot s \cdot m^2$.

In North America, much emphasis is placed on detailing to reduce the exposure of the bales to moisture (Magwood *et al*., 2005): (1) a minimum roof overhang of 600 mm for protection from wind-driven rain is recommended; (2) isolating the straw from the foundation by installing vapour barriers between the foundation and bales, and providing flashing at the base on the exterior wall is recommended; and, (3) the foundation design should ensure the bales are 200 mm to 300 mm above grade. It should be noted, however, that such details are based on the experience of builders of low-rise residential construction, and may not be appropriate in other cases. There is, to date, little quantitative research to help designers understand the behaviour of straw bale walls subjected to moisture.

A study by Jolly (2000) of 18 existing straw bale houses in Canada located in regions subjected to mild to moderate amounts of precipitation showed that the majority had moisture contents in the straw bale walls of 14% or less.

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Four of the houses were deemed to have unacceptable straw moisture levels, and these were attributed to these houses having poor detailing, such as non-existent roof overhangs, or bales located below grade.

The low moisture content observed by Jolly (2000) is significant, as work by Summers (2005) and Summers *et al.* (2002) indicated that rot in straw does not initiate until at least 30% moisture content. The authors recommend that bales with moisture content greater than 25% should not be used for construction.

Fire testing

Fire safety remains one of the common concerns working against straw bale construction, in public opinion. However, limited fire tests done on plastered straw bale walls suggest that they are resistant to fire damage, flame spread and combustion. The key is that the bales are protected by a thick coat of plaster, which acts as an insulator. Precautions must be taken during the construction of a straw bale building before plaster is applied.

Two full-scale straw bale walls were fire tested in 2006 according to ASTM E119 (King, 2006). An earth plastered wall was exposed to a one-hour fire test. The second wall had a lime-cement plaster and bales on edge so that the bale strings were exposed just under the surface of the plaster. It was exposed to a two-hour fire test. The ASTM E119 test involves exposing one side of the wall to a maximum temperature of 927°C. Temperature on the unheated side is monitored to ensure it does not exceed the threshold specified by ASTM E119. Both walls passed this standard test.

Case study – the 4C's food bank and thrift store (adapted from Vardy *et al.*, 2006)

Approximately 100 straw bale buildings have been constructed in Canada in the past 10 years. Most of these are residential structures, but more recently structures with public access have been successfully designed and constructed. For example, a thrift store and food bank was constructed in 2005 in Haliburton, Ontario (located approximately 230 km north-east of Toronto). The construction was carried by students as part of a course in sustainable building at Sir Sandford Fleming College.

The building is designed with a simple rectangular footprint encompassing approximately 170 square metres of interior floor space. As shown in **Figure 10**, this space was subdivided to separate rooms for a thrift store retail area, a food bank area, a small office and sorting room, and a number of smaller rooms for various purposes.

The exterior walls of the structure were designed as load-bearing plastered straw bale walls. This system was chosen as it met the owners' desire for an environmentally sustainable design and because the ease of construction of plastered straw bale walls allowed the students in the course the opportunity to construct the walls themselves. Locally grown hemp bales were used for the straw in the walls, while locally obtained clay/silt was use in the earthen render. A mix design of two parts sand to one part clay/silt was chosen for the majority of the wall thickness. Some mixes also included cattail heads and/or chopped straw as fibre reinforcement. The plaster was hand-applied to a minimum thickness of approximately 38 mm. It should be noted that the use of earthen plaster may not be appropriate for all climates.

Because of the lack of knowledge at the time of construction regarding the structural performance of earthen rendered straw bale walls, the structural engineer of record required test data. Of concern were the large snow loads that can occur in this part of Canada. The 2.44 m × 2.44 m specimen obtained a maximum load of approximately 32 kN/m, which far exceeded the required value of 23.5 kN/m.

In addition to the unique earthen rendered straw bale walls, the structure also contains a number of different alternative and environmentally sustainable construction materials and techniques throughout the roof, foundation, interior walls and other components of the building. These include, but are not limited to:

- compressed straw panel interior walls;
- infill interior walls utilising a variety of infill materials such as plastic bottles and hempcrete;
- earthbag stacked footing on rubble trench foundation;
- minimum of 85% recycled paper content in cellulose roof insulation;
- finished earthen floor;
- hydronic radiant floor heating with solar pre-heating;
- solar domestic hot water heating;
- electricity provided by 1 kW grid inter-tied photovoltaic (PV) array;
- fibreglass windows located in the exterior walls;
- 'sun tubes' installed through the roof structure;
- low-energy lighting and electrical fixtures and appliances;
- finishes chosen for minimal toxicity and optimal functionality.

The 4C's project is a unique project for a number of significant reasons. The gravity loads expected to act on the earthen rendered straw bale walls are significantly greater than would be encountered on a residential construction project. The energy efficiency of the structure ensures low operating costs for the charity organisation, while the simplicity of construction enabled the students in the Sir Sandford Fleming College programme to participate as much as possible in the construction.

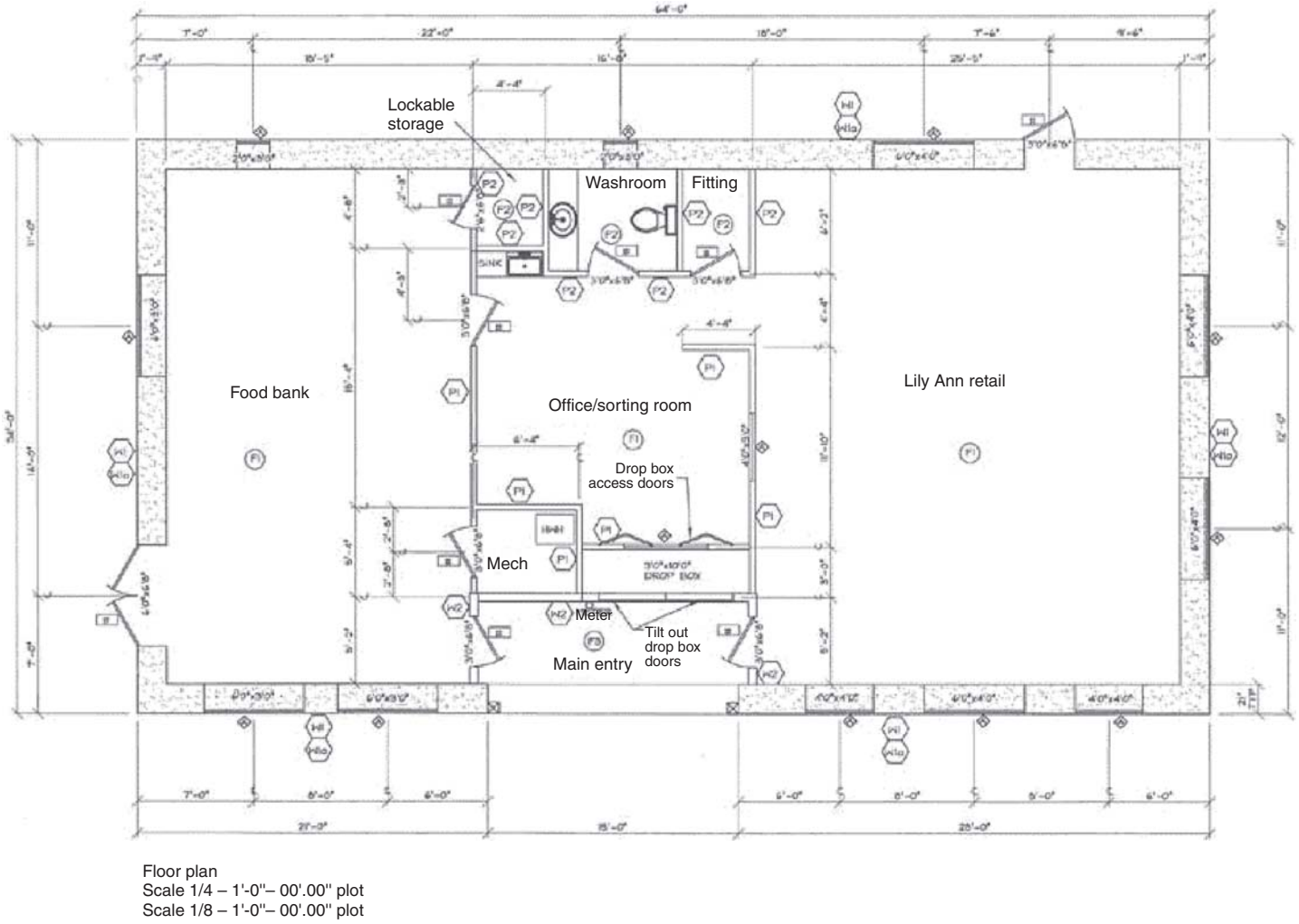

Figure 10 Floor plan of 4C's building (courtesy of Chris Magwood)

The future of straw bale construction

Straw bale construction has made great strides in the past 10 years in terms of increased understanding of this construction technique. There are now numerous examples of successful use of this technique in a variety of climates. In many respects, straw bale construction has entered the mainstream of residential construction.

However, there are still many barriers that exist for straw bale construction. There is a lack of standards and building codes that can make it difficult to get approval for straw bale construction. Belarus and Germany both have limited straw bale building codes (King, 2006). In the US, a straw bale code was adopted in Tucson/Pima County, Arizona in 1996 (King, 2006). A straw bale code is currently being considered for inclusion as an appendix to the 2007 California Building Code (King, 2006). However, the more typical situation is that in Canada. To obtain a building permit, it is often required to have the design reviewed by a professional engineer. Because of a lack of codes and design information, engineers may require structural testing on innovative wall designs. This is a time-consuming and expensive endeavour, and the results, although of great significance to the specific project, are difficult to apply to other projects where a different wall design may be used. This is a significant issue, as there is currently very little consistency between wall designs from project to project due largely in part to a lack of building codes for straw bale construction.

In general, any research on the structural performance of plastered straw bales and straw bale walls would be beneficial to the straw bale building community, as it has been noted that there are numerous gaps in the current knowledge. More specifically, there is a need for more experiments conducted under controlled conditions, which produce repeatable results in order to gain a greater fundamental understanding of the behaviour of plastered straw bale walls. Once the behaviour of laboratory specimens is

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understood to its fullest extent, a transition must be made to correlate this understanding to an understanding of more realistic, site-built walls containing imperfections in dimensions and variability in material properties. This understanding is critical for bringing a modern limit-states design approach to straw bale construction.

With this type of understanding, architects and engineers may be willing to consider this type of construction, and its environmental benefits, for larger projects. To date, the vast majority of straw bale construction has been for single-storey residential applications. However, it has been by the work of Magwood *et al*. (2005) that straw bale construction has come to be recognised as ideal for many industrial applications. These types of buildings are typically large 'boxes' with very few openings. Especially in projects where the building requires significant insulation, straw bale could be a much more environmentally friendly means of constructing these buildings than the conventional concrete or steel solutions. A recent, and promising, development has been the use of modular straw bale panels. The Modcell company in the United Kingdom fabricates panels under controlled conditions in a factory and transports them to construction sites for assembly (*BD Magazine*, 2007). This of course may increase the embodied energy of the panels. However, it does address concerns regarding wall variability, moisture, and constructability. These panels were used, for example, in the 1129 m^2 York EcoDepot.

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Chapter 74

Alternative materials for construction

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Recent environmental issues worldwide have been attracting interest in the use of alternative materials such as natural fibre composites that are environmentally friendly. The use of waste plastic to develop composites materials is a further advantage in promoting environmental sustainability. In this work, hemp fibres were used to reinforce waste agricultural bale wraps to develop composites materials that will be suitable for many low-cost applications. The use of hemp fibres to upgrade waste agricultural bale wrap helps to improve the mechanical properties of the waste plastic. The mechanical and physical properties of the composites are affected by several factors such as processing methods, properties of the waste plastic, fibre volume fractions and orientation of the fibres. The mechanical properties of the composites are, however, less than those of synthetic fibre composites; further improvement can be obtained through proper processing of the fibres, waste plastics and the composites. The composites materials can be suitable for many low-cost applications in the building and construction industries such as ceiling tiles, floor deck, handrail, furniture, doors and wall panels.

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Introduction

The use of polymers in many engineering applications requires that their strength and stiffness be enhanced to withstand stress. One way to achieve this is by the addition of reinforcing fillers in the form of fibres to create composites with higher strength and stiffness. Fibres used as reinforcement for polymers are mostly synthetic fibres such as glass, carbon, Kevlar, aramids, etc. (Hull, 1981) and their composites have long been used in many engineering applications, aircraft and construction industries. Composites offer a combination of properties not achievable by either component alone; lightness, resistance to corrosion, resilience and high strength-to-weight ratio. In the last two decades interest in natural fibres as replacement for synthetic fibres (glass fibres especially) in polymer composites has been growing rapidly. This is due to an increasing environmental awareness, health hazards associated with the manufacture and use of glass fibre composites, and the fact that the composites are non-recyclable.

Natural fibres such as hemp, wheat, corn, wood, etc. are cheap, renewable, recyclable, abundant and biodegradable resources. Combining natural fibres with polymers creates materials that are lighter in weight, cheaper, have good strength-to-weight ratio, and are biodegradable and recyclable. Furthermore, the use of waste polymer as a matrix source in composites promotes environmental sustainability by reducing the amount of waste plastics going to landfill, and also by reducing the pressure on the use of virgin polymers. However, the high moisture sensitivity of plant fibres, thermal stability of plant fibres during composite processing and difficulty in fibre processing continue to be a challenge to the development of plant fibre composite materials (Karmaker, 1997).

Source of natural fibres

Natural fibres used as reinforcement in composite materials are polyssacharide and proteinous materials obtained from both plant and animal sources. The fibres are either extracted from the stem, leaf, seed of plants, or from animal wool/hair (Nishino, 2004). The plants from which fibres are obtained include corn, flax, sisal, kenaf, cotton, wood, etc. A grouping of the various plant fibres and the part of the plant from which they are obtained are shown in **Figure 1**. Plant fibres are subdivided into wood and non-wood plant fibres. Non-wood plant fibres are classified by reference to the part of the plant from which they are extracted, namely straw fibres, bast fibres (fibres obtained from the stem of the plant), leaf fibres, grass fibres, and seed and fruit fibres. The most important types of non-wood plant fibres used in composite materials are flax, hemp, jute and kenaf which are extracted from the stem of the plant. Animal sources for fibres include avians, sheep, insects, and examples of their fibres include wool, feather, silk and spider silks (Martinez-Hernansez *et al.*, 2005; Zhang, 2002).

Figure 2 shows photographs of hemp plants (*Cannabis sativa L.*) and fibres extracted from hemp stems. The structure of a typical hemp stem consists of a hollow wooden core known as xylem that is surrounded by growing tissues called cambium, an outer ring of phloem, cortex, epidermis and cuticle (Fortenbery and Bennet, 2004). The phloem and cortex are the fibres which give the hemp stem the required

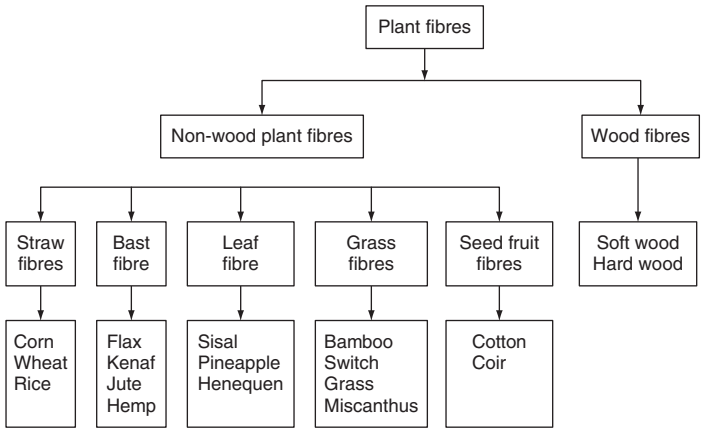

Figure 1 Classification of fibres which can be used as reinforcement or fillers in polymers (adapted from Mohanty *et al*., 2002)

strength. The fibres, like other fibres in the same category, are multicellular structures whose major compositions are cellulose, hemicellulose and lignin. **Table 1** illustrates the mechanical properties of some plant fibres. The values are scattered widely and are much lower when compared to the properties of glass fibres. However, because of their low density, the specific properties (property-to-weight ratio) of some fibres, e.g. hemp, are comparable to the values of glass fibres. The mechanical properties of the fibres are greatly influenced by several factors such as crystallinity, microfibril angle, source, age, retting and extraction processes (Nishino, 2004).

Sources of waste plastic

Waste plastic or post-consumer plastic are plastic materials discarded after completing their service time. The waste plastics come from a wide variety of sources and consist of mostly thermoplastic resins and some thermosetting resins. The sources of waste plastic can generally be grouped into household and industrial sources (**Table 2**). The largest fraction of waste plastics consists of polyolefin such as polyethylene (high-density polyethylene (HDPE), low-density polyethylene (LDPE) and linear low-density polyethylene (LLDPE)) and polypropylene (PP). The remaining components include polystyrene (PS) polyvinyl-chloride (PVC) and polyethyleneterephtalate (PET) (Harper, 2002).

The main problem being addressed in many municipal thermoplastic recycling facilities is the heterogeneity of waste plastics: several different polymers are present in plastic wastes and many of them are incompatible. It is practically impossible to reprocess the waste plastic as a whole, and therefore separation of the plastics according to their different resins has become imperative. Industrial sources of waste plastic are however more easily controlled, as the waste plastic is usually of one resin type, or in some cases the manufacturer ensures that various resins are separated according to their type before being sent for recycling.

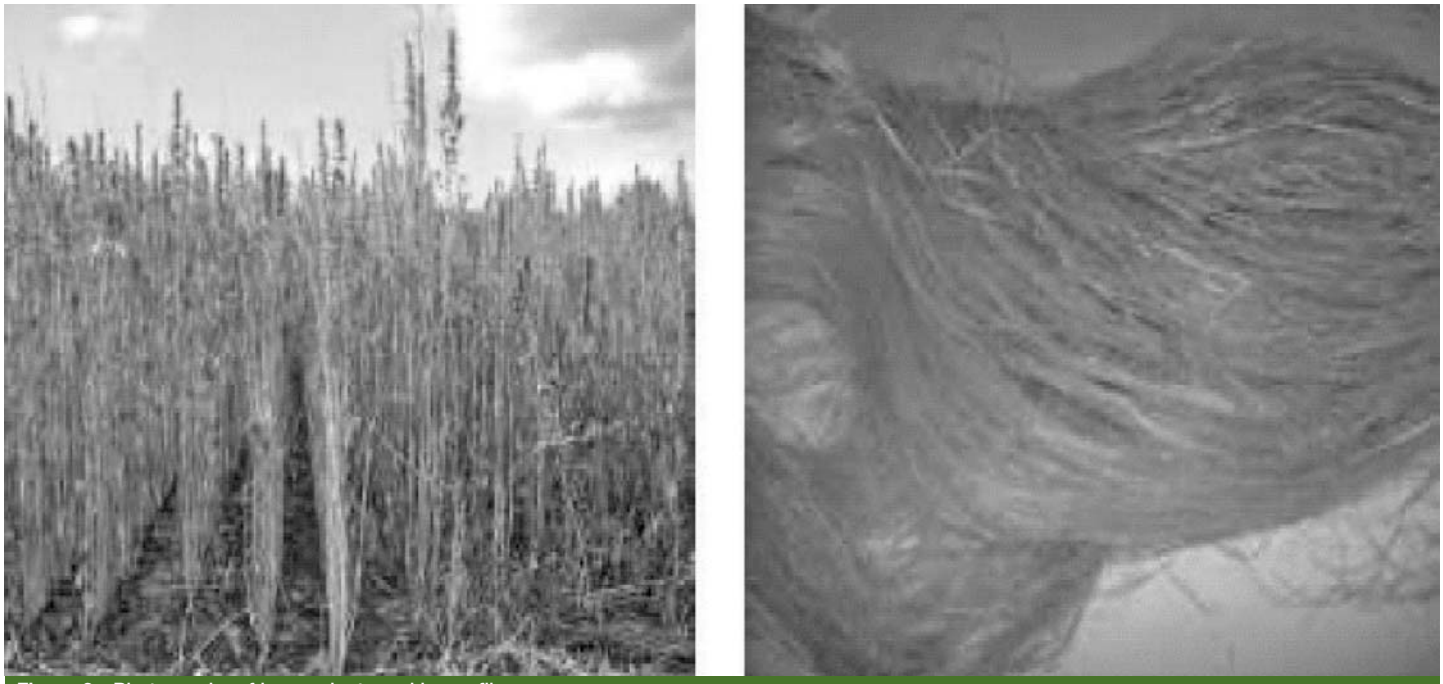

Figure 2 Photographs of hemp plants and hemp fibres

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Plant	Fibre type	Density: g/cm^3	Tensile strength: MPa	Young's modulus: GPa	Elongation at break: %	Specific tensile strength: GPa per g/cm^3	Specific modulus: GPa per g/cm^3
Hemp	Bast	1.47	690	70	1.6	0.47	47.6
Flax	Bast	1.50	345–1500	27.6	2.7–3.2	0.23–1.0	18.4
Jute	Bast	1.3–1.45	393–800	13–26.5	1.16–1.5	0.30–0.62	10.0–20.4
Ramie	Bast	1.5	400–938	61.4–128	1.2–3.8	0.27–0.63	40.9–85.3
Sisal	Leaf	1.45	468–640	9.4–22.0	3–7	0.32–0.44	6.48–15.2
Cotton	Seed	1.5–1.6	287–800	5.5–12.6	7.0–8.0	0.19–0.53	4.67–5.33
Coir	Fruit	1.15	131–175	4.0–6.0	15–40	0.11–0.15	3.48–5.22
E-glass		2.55	3400	73	2.5	1.33	28.6

Table 1 Physical and mechanical properties of some plant fibres (adapted from Bismarck *et al*., 2005)

Domestic waste in most developed countries is also collected and sorted. However, agricultural wastes present a serious ecological problem. Waste bale wrap, for example, is dirty and mixed with other materials such as sand, weed and paper (**Figure 3**). The quality of all recycled plastic is dependent on the history of the polymer synthesis, primary processing and application, composition, additives, residual stabiliser content and extent of contamination by impurities (Pospisil *et al*., 1995).

Development of hemp fibre/bale wrap composites for construction

Agricultural bale wrap is a linear low-density polyethylene (LLDPE) polymer used by farmers for storage of agricultural products to reduce the risk of weather damage after harvesting and to mulch crops. The use of agricultural bale wrap by farmers worldwide has increased rapidly in recent years. Most of the agricultural bale wrap is not collected at the end of its use and is often treated by incineration, landfilling or buried in the farmers' fields. The increase in the volume of waste plastics being put into landfill or burned either at the site of generation or at local waste management facilities has attracted interest in the use of recycled waste plastic from both the public and private sectors (Harper, 2002; EPIC, 2007). The waste polymers are often degraded as a result of photolysis with ultra violet (UV) light during their service time (Hodzic, 2004; La Mantia and Gardette, 2002). This can give rise to poor mechanical properties of the recycled material (La Mantia *et al*., 1996).

A good way of improving the properties of recycled plastics is to add fibres and to create upgraded composite materials. The addition of fillers to waste plastics during processing can help to improve many of their properties: impact strength, elastic modulus, flexural properties, etc. (Dintcheva *et al*., 1997). Upgrading waste agricultural bale wrap with fillers such as hemp fibres can be an efficient way to recycle waste bale wraps and provide the desired

Waste plastic source	Kinds of waste plastic	Resin type
Household	Grocery bags, beverage bottles, cosmetic bottles, food containers, plates, cups	LDPE, LLDPE, HDPE, PET, PS
Automotive	Bumpers, battery, door linings	PP, ABS, PVC
Packaging	Cushioning materials, containers, bottles	PS, PVC, PP
Agriculture	Hay bale wrap, greenhouse films	LDPE, LLDPE
Electronics	Computer, radio, telephone, television	PP, PVC, ABS, PS, PE
Construction	Pipes, insulators, bags	PVC, PS

Table 2 Some examples of waste plastic sources and plastic types

Figure 3 Waste agricultural bale wraps collected from local farmers

solution to the waste problems. Hemp has promising properties which, when combined with waste plastics, can result in materials with high mechanical and physical properties. Hemp fibre is one of the strongest and stiffest plant fibres (Pickering *et al.*, 2007). Using hemp fibres to reinforce waste agricultural bale wraps will create composite materials that are environmentally friendly, lightweight, biodegradable and cheap. Being lightweight materials, the hemp fibre composites can be very cost-effective for a number of applications in the building and construction industry, transportation industry, and a number of semi-structural applications. The composite materials can be used for making car door panels, panels for partitions, false ceilings, window and door frames, roof tiles, and mobile or prefabricated buildings which can be used in times of natural disaster. Further uses in the construction industry include construction of floor decks and handrails, where they serve a good replacement for wood materials.

Harvesting, retting and extraction of hemp

Hemp is an annual long-fibre plant, and commonly has male (staminate) and female (pistillate) plants growing separately. It can be cultivated in a wide range of environmental conditions ranging from temperate to equatorial climates. The cultivation of hemp usually begins in April–June, particularly in temperate regions such as the United States, Europe and Canada. This is the most effective mechanism for planting noted, which leads to optimum fibre yield and quality (Struik *et al.*, 2000; Harbans *et al.*, 2005). The cultivation period of hemp (seeding to harvesting) usually takes three to four months. Harvesting of hemp depends on whether the plant is grown for high-quality fibre or for seed. When grown for seed, the plant is usually harvested when the seed is mature. Harvesting of hemp grown for fibre is best before seeds are fully matured to produce optimum quality fibre, i.e. at mid-flowering stage (Merfield *et al.*, 1999). This is usually from August to September depending on the weather conditions. Harvesting is usually carried out by cutting, pulling, or using a tractor-drawn harvester.

Figure 4 show a simple preparatory route for hemp fibre from harvesting stage to the fibre stage. Once harvested, the stems are often field retted to break the chemical bonds that hold the bast fibres together. Field or dew retting is done by simply spreading out the harvested stems in the field where indigenous aerobic fungi grow on the plant and partly degrade the polysaccharides in the middle lamellae, thus releasing the fibres for further processing. However, the challenge associated with dew retting includes microbiological contamination of the fibre, increased variability of fibre properties, and insecurity of fibres supply owing to changing weather conditions during field retting. To make retting more controllable, a process known as 'water retting' in which the plant stems are retted in a water tank or still or slowly flowing body of water is used (Van Sumere, 1992).

Other processes such as steam explosion and enzyme retting have been developed to separate the fibres from the hurds (Van Sumere, 1992; Mackie *et al.*, 1985). These processes are, however, capital intensive and there are environmental issues arising from the use of water retting process. For economic reasons, field retting is still widely adopted for pretreating fibres. After the retting process, the dried stalks are bailed and sent to a process mill for scutching and decortication. Scutching is the process of breaking the woody core of the stems into pieces, while decortication is the process of separating the bast fibre from the hurd (Fortenbery and Bennet, 2004). The final processing steps of the fibre depends on the application whether for textile or composites or for any other application.

Waste plastic preparation process

The preparation of waste plastic for further reprocessing consists of several phases of activity: collection, separation and processing (**Figure 5**). Processors of waste plastics have developed these activities as a general procedure to obtain quality feedstock for reprocessing. The steps have allowed for effective waste plastic processing, resulting in a quality product at the end of the recycling process. The final reprocessing steps of the waste plastics depend on the use of the final plastic products.

Waste plastic is often converted into plastic lumber, used for making plastic containers, carpets and fibres. The waste plastic can also be sent through a chemical recycling process in which the waste plastics are converted into the original monomers used to make the plastics (Arnold, 2004). The adoption of all the stages of activity in **Figure 5** depends on whether the waste plastics are obtained from municipal or industrial sources. Municipal sources are usually a mixture of several kinds of polymers and the preparation process requires adoption of all the stages of activity. Waste plastics from industrial sources are in many cases made of a single resin type. Such process makes the preparation of waste plastic less tedious and less expensive.

In **Figure 6** the preparation of waste agricultural bale wraps collected from local farmers in Kingston, Ontario for use in composite processing on a laboratory scale is shown. It consists of a simple inspection, cleaning and drying process. Such a process can be adopted easily to prepare waste plastic films for further reprocessing and on an industrial scale washing and drying machines would be utilised.

Fabrication of composites

The identification of optimum processing methods for composites is clearly an essential step in the development

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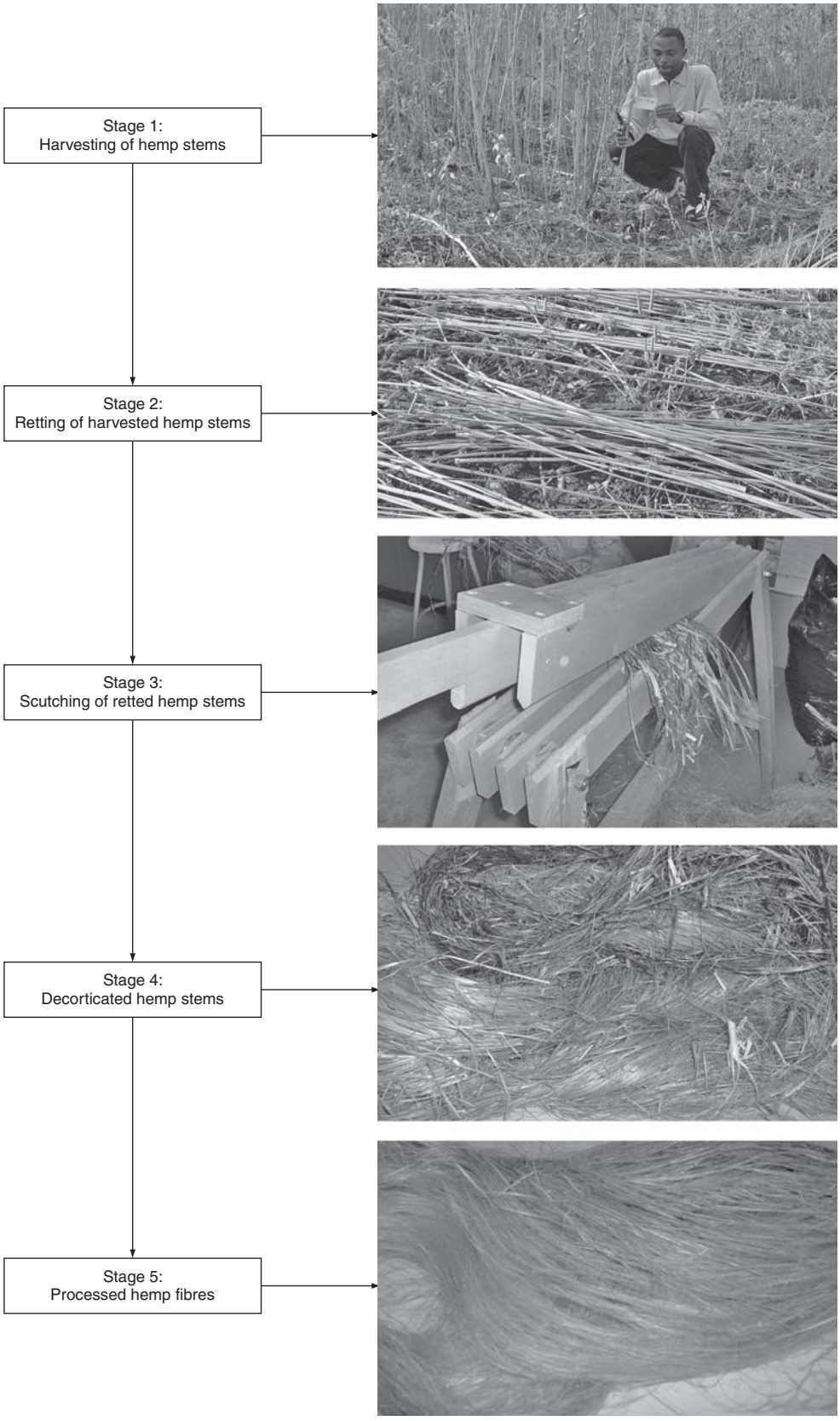

Figure 4 Hemp fibre processing route

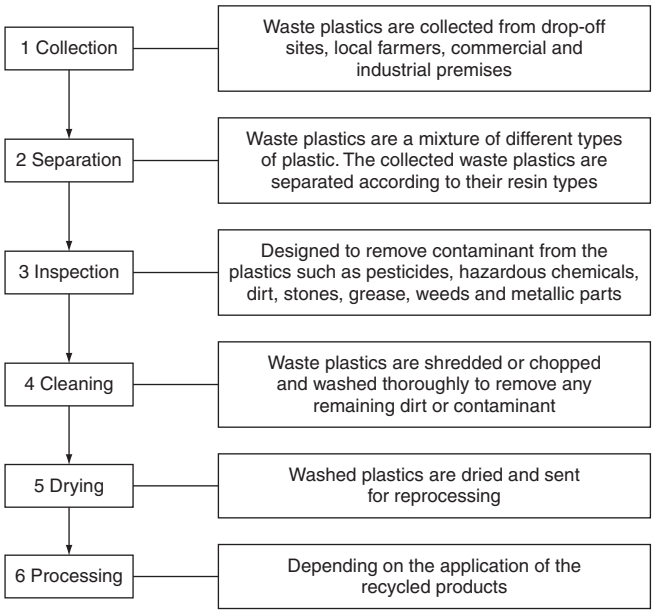

Figure 5 Waste plastic preparation process

of these products. There are various techniques for the manufacture of fibre-reinforced thermoplastic polymers. Processing of plant fibre/thermoplastic composites can be, for example, by extrusion, injection moulding or compression moulding. **Table 3** gives a brief description of the manufacturing process of composites. Plant fibre thermoplastic composites can be manufactured by extrusion followed by injection or compression moulding (Karmaker and Youngquist, 1996; Oksman, 2000). In the process, the fibre lengths are significantly reduced due to shear mixing of the composite by the extrusion process. The orientation of the fibre in the composites in this process is random (three dimensional). Composites can also be manufactured using a film-stacking method followed by compression moulding (Garkhail *et al.*, 2000; Madsen and Lilholt, 2003). Fibre mats are usually arranged in alternate layers with plastic film before compression moulding. The orientation of the fibre in the composites can easily be controlled in this process. However, the degree of impregnation of the matrix into the fibres is not as efficient as using the extrusion process. Thermoplastic pultrusion processes have also been used in the manufacturing of plant fibre/thermoplastic composites (Van de Velde and Kiekens, 2001). This process is, however, not very common. The most important criterion in the processing of the composite is to ensure proper mixing between the fibre and matrix such that individual fibres are completely encapsulated inside the matrix material.

In **Figure 7** a typical laboratory processing of hemp fibre/waste LLDPE composites is shown. The process consists of compounding of the fibre and matrix using a single screw extruder followed by compression moulding of the extruded pellets to form a composite sheet.

Mechanical properties of hemp fibre composites

Agricultural bale wrap (LLDPE) has low mechanical properties. The degradation of the waste bale wrap during usage (UV degradation) further reduces the properties. The addition of hemp fibre to the waste bale wrap can lead to property improvement. However, the main problem in processing hemp fibre/waste LLDPE composites includes variations in the quality of the raw materials, incompatibility between the hydrophilic hemp fibre and the hydrophobic LLDPE matrix, thermal stability of the cellulose fibres during processing and susceptibility of the composites to moisture absorption due to the presence of hemp fibres. Many desired properties can, however, be achieved through proper processing of the composite. A number of researchers have investigated the reinforcement of thermoplastic polymers with natural fibres such as hemp, flax, jute, etc. A general finding is that the mechanical properties of the composites are highly dependent on:

- fibre physical and mechanical properties
- method of manufacture of the composites
- fibre orientation within the matrix
- proportion of fibre to matrix.

The properties of the hemp fibre/waste LLDPE will depend on the properties of the constituent materials. The main function of the hemp fibre is to carry the majority of the load applied to the composites, which in turn will depend on the strength and stiffness of the fibres. On the other hand, the properties of hemp fibres used as reinforcement are influenced by the cultivar, growth and processing condition of the fibres. Proper processing can yield fibre with good mechanical properties leading to improved composite performance (Hornsby *et al.*, 1997). Another important property of the fibre that can affect the composites is the size of the fibre (fibre length). The length of the fibre should be above a critical fibre length for effective transfer of stress from the matrix to the fibre (Pervaiz *et al.*, 2005).

Table 4 gives the mechanical properties of hemp fibre/waste LLDPE composites manufactured with different techniques. The high shear mixing of the extrusion process leads to reduction in length of the fibre and their reinforcing effects. On the other hand, it can lead to proper impregnation of the matrix into the fibre which is beneficial for the environmental durability (moisture absorption) properties of the composites (**Figure 8**). Composites fabricated using hand lay-up and compression moulding techniques are less susceptible to shear deformation of the fibre. In longitudinal and transverse oriented composites, the composites possess orthotropic

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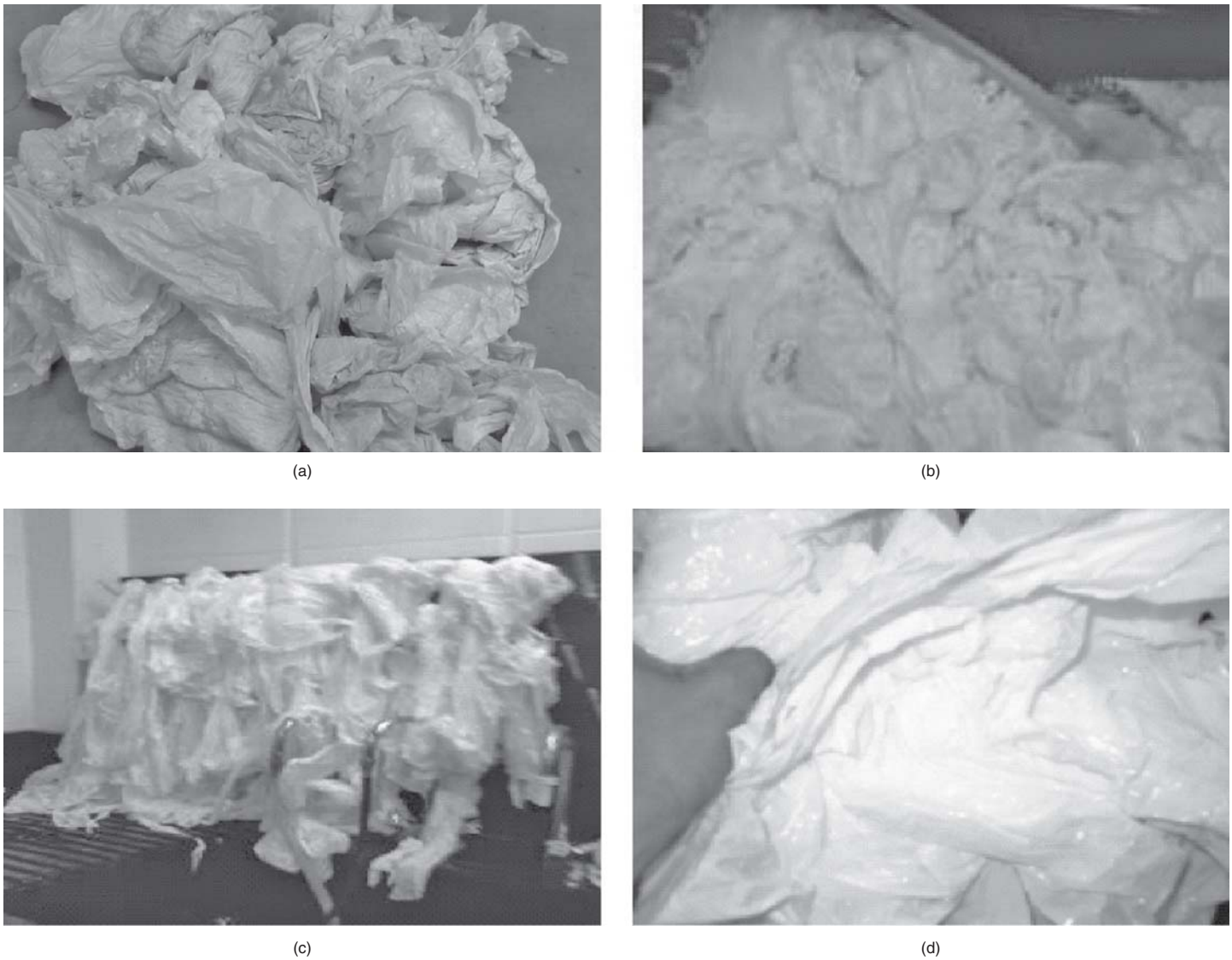

(a) (b) (c) (d)

Figure 6 Preparation of waste agricultural bale wrap for further processing into composite materials in the laboratory: (a) waste bale wrap materials are shredded; (b) waste bale wrap are washed; (c) drying of washed bale wrap; (d) cleaned and dried bale wrap

Fabrication technique	Description
Extrusion	Used to create long objects or profiles. A compounded mixture of fibre and thermoplastic polymers in the form of pellets is fed into a heated chamber of the extruder and carried along by a feedscrew. As it is conveyed it is melted in the chamber, plasticised by the screw, and continually forced out of the chamber through a die of given configuration. The extrudate is cooled resulting in resolidification of the composites, after which the composites are cut into lengths
Injection moulding	Used to make products of intricate shape especially for the automotive industry. Unlike the extrusion process, the compounded material is heated and pumped into a mould of the desired shape of the products. The mould is then open and the finished product is removed
Compression moulding	Prior to compression moulding, the fibre and resin are thoroughly mixed using a mixer or an extruder. The semi-prepared composite material to be moulded is placed in the mould cavity and the heated platens are closed by a hydraulic ram to apply pressure to the materials. The material flows and fills the mould to form the desired shape
Pultrusion	In the pultrusion process, continuous strands of fibre reinforcement are impregnated with resins in a resin bath and drawn through a heated die to form composite materials with constraint cross-section. The cooled and solidified composite materials are then cut to the desired length

Table 3 Fabrication methods of fibre/thermoplastic composites

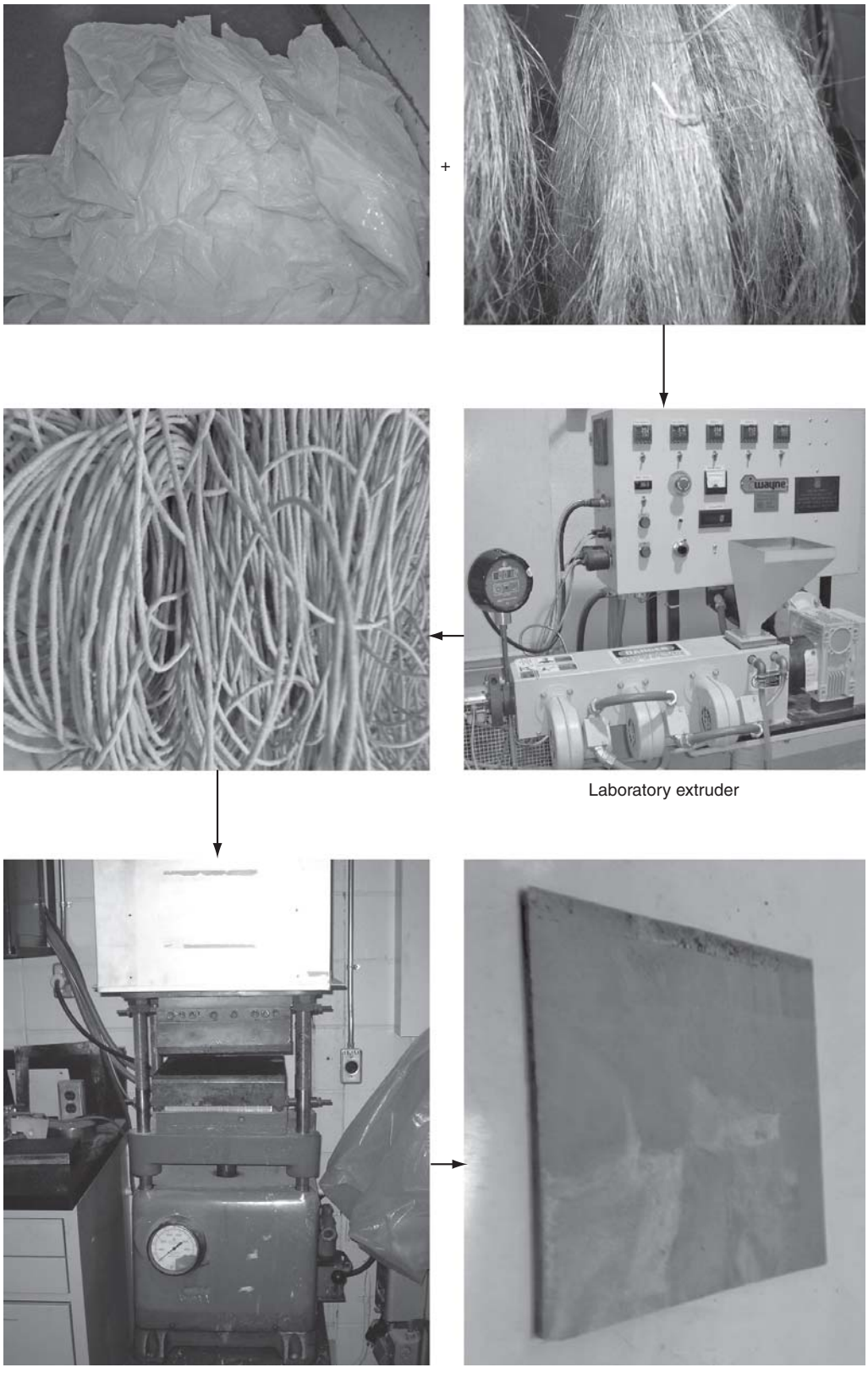

Figure 7 Preparation of hemp fibre/waste bale wrap composites in the laboratory

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Fabrication/orientation method	Filler content: %	Tensile strength: MPa	Tensile modulus: MPa	Flexural strength: MPa	Flexural modulus: MPa
Longitudinal/hand lay-up + compression	20	37.6	1438	12.9	537
Transverse/hand lay-up + compression	20	4.44	87.9	–	–
Random/extrusion + compression	20	10.0	284	15.1	936
Random/hand lay-up + compression	20	11.5	382	18.6	761

Table 4 Properties of hemp fibre/waste LLDPE composites prepared by different processing techniques and different fibre orientation

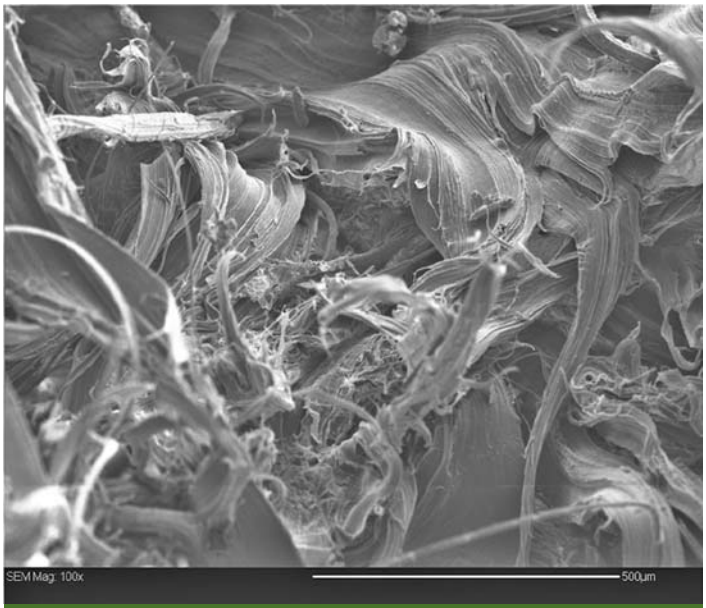

Figure 8 Fracture surface of hemp fibre/waste LLDPE composites showing good impregnation of hemp fibre by the LLDPE matrix

properties with the greatest stiffness and strength in the direction of the fibres. **Table 5** illustrates the reinforcing effects of hemp fibre on the waste LLDPE matrix.

Moisture-resistance properties of hemp fibre composites

The application of hemp fibres as reinforcement in polymer matrices is limited because of the susceptibility to moisture absorption. Hemp fibre contains large OH^- group in the cellulose and hemicelluose components which can easily form hydrogen bonds with water molecules leading to swelling of the fibre and creating dimensional instability in the composites. Degradation due to moisture absorption can significantly reduce the life span of plant fibre composite materials. Moisture absorption can cause changes in the strength, stiffness and strain to failure of the composites. The absorbed moisture can act as a plasticiser and causes cracks in the matrix, leading to fibre–matrix debonding and degradation of the fibres. This will ultimately result in the reduction of mechanical properties of the composite materials and effective lifetime of the composites (Dhakal *et al.*, 2007).

Studies have reported that the movement of water molecules into the composites can be caused by several mechanisms (Espert *et al.*, 2002; Karmaker *et al.*, 1997):

- Diffusion of water molecules into the micro gaps in the polymer chain.
- Capillary transport into the gaps and flaws at the interfaces between fibre and polymer.
- Flow of water into the microcrack or pores arising from the swelling of the fibre inside in the composite structure.

The above processes results from imperfect processing of the composites material, improper wettability or impregnation of the matrix into the fibre, weak interfacial adhesion, or the service conditions of the composites materials. The diffusion of water molecules into the composite materials can also depend on temperature, humidity, fibre volume

Orientation/fabrication method	Fibre content: %	Tensile strength: MPa	Tensile modulus: MPa	Flexural strength: MPa	Flexural modulus: MPa
Random/extrusion + compression	0	8.09	144	8.08	301
	20	10.0	284	15.2	936
	25	9.32	314	15.7	963
	30	9.64	422	15.7	887
Longitudinal/hand lay-up + compression	15	18.0	516	–	–
	20	37.6	1438	12.9	537
	25	32.3	1404	16.0	807
	30	–	–	18.7	993

Table 5 Mechanical properties of hemp fibre/waste LLDPE composites with different fibre content

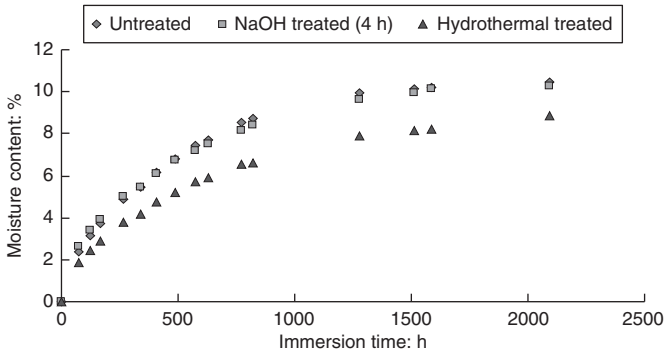

Figure 9 Moisture content as a function of immersion time for treated and untreated hemp fibre/recycled LLDPE composites

fraction, presence of voids in the composite materials and viscosity of the matrix (Dhakal *et al*., 2007). The moisture sensitivity of most plant fibre composites can, however, be improved through proper processing of the composites and fibres. **Figure 9** shows the plot of moisture absorption of treated and untreated hemp fibre/waste LLDPE composites manufactured by the extrusion and compression moulding process. Proper treatment of the fibres reduces their sensitivity to moisture absorption by reducing or blocking the OH^- groups available for bonding with water molecules. This consequently increases the moisture resistance properties of the composites. The maximum moisture content and diffusion coefficient of the hemp fibre composites after 8 weeks in water is shown in **Table 6**. The details of the diffusion coefficient of the composites may be found in Aghedo (2007).

Examples of building products

The use of plant fibres to reinforce thermoplastic materials gives designers and engineers new options and solutions for product development. The advantages of the low-cost, renewability and worldwide availability of the fibre and matrix sources, together with their environmental benefits, present an attractive future for this class of material. Natural composite materials are used in a wide range of products ranging from low-performance composites to structural components in the building industry. The lower mechanical properties of natural fibre/waste plastic composites indicate that these materials cannot be used for structural applications

Samples	Saturation moisture, Mm: %	Diffusivity, D: mm^2/hr
Untreated	10.47	1.81×10^{-3}
NaOH treated	10.25	1.69×10^{-3}
Hydrothermal treated	8.85	1.49×10^{-3}

Table 6 Saturated moisture content of hemp fibre/waste LLDPE composites

Figure 10 Furniture fabricated using recycled plastic material

until further research is done to improve the properties. However, these materials have huge undeveloped existing potential in low load-bearing applications in the building industry, namely for ceiling tiles, wall frames, floor deck, furniture, railings, doors and window frames.

Figure 10 shows an example of furniture manufactured using recycled plastic materials. The addition of fibres such as hemp will help to improve the stiffness of the products.

Conclusion

There is great potential for composite materials from natural fibre and polymer matrix as a substitute for synthetic fibre composites in many applications. They are cheap, lightweight and environmental friendly materials. The use of waste plastic is a significant advantage in reducing deteriorating environmental conditions in many urban cities, especially in the developing countries. Fibres such as hemp, jute and flax can be used as substitutes for synthetic fibres in developing composites for many applications. A simple processing route can be adopted for processing natural fibre composites without huge financial outlay. The mechanical properties of natural fibre composites are less than those of synthetic fibre composites; however, their strength-to-weight ratio is comparable or higher than those of synthetic fibre composites. This advantage makes natural fibre composites more attractive to material scientists, designers and engineers for developing low-cost materials. The composites can be used for the manufacture of products for the building, furniture, transportation and packaging industries.

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Chapter 75

Architectural fabrics

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Architectural fabrics are an integral part of *fabric architecture*, a term used to describe tensile fabric membranes – also known as fabric structures or textile buildings. This chapter discusses architectural fabrics in the context of their physical properties, durability, aesthetics and suitability for a given application. The account is preceded by a brief background to fabric structures and related design issues. Images of samples of different types of architectural fabrics, their descriptions and examples of application in real projects are used to clarify the subject. There are numerous factors affecting the choice of architectural fabric and these include a manufacturing design stage, known as *patterning* – the creation of a tensile surface in three-dimensional space out of a series of two-dimensional, unstrained pieces of fabric. Patterning is presented in a concise manner, sufficient to highlight design issues, together with problems of choice of fabric and behaviour in a full-scale tension structure. The chapter ends with a brief discussion of factors influencing the choice of architectural fabric, future research trends and development of new materials.

Introduction

Architectural fabric is used to form a membrane surface in a tensile roof or an architectural enclosure. The fabric membrane is held in tension by supporting elements, such as cables, columns, masts and beams. The cables are used to create ridges, valleys, edge boundaries and tie-backs. Like the fabric membrane, they work in pure tension. The rigid elements, in the form of columns, masts and beams, provide support to the membrane by developing compression and bending actions. A good introduction to tensile roof structures is given in Shaeffer (2006) and Huntington (2003). The use of a broad variety of materials in innovative surface structures is discussed by Bechthold (2008).

The popularity of fabric structures is due to their potential to offer freedom of artistic expression in terms of providing visually light and aesthetically pleasing structural forms. If properly designed, fabric structures are very efficient in terms of using a minimum amount of material while maintaining maximum stability and strength. Their exploitation potential is limitless, as exemplified by roofs over sports stadia and exhibition venues, canopies in shopping malls and seasonal shades. Tent-type structures can be used in the areas of disaster relief.

The prestress in fabric membranes can be achieved by applying a tension force at the boundaries of the structure, or by supplying an air-pressure within a fabric enclosure. The latter generates structures known as air-houses, pneumatic membranes, or cushions. These constructions require the use of very light fabrics, such as foils (discussed under ETFE foils).

Current design codes for conventional materials are based on the limit state philosophy, with partial factors of safety applied to loading and material strength. For fabric structures, this philosophy does not apply, because margins of uncertainty with regard to material properties are too great. Furthermore, the geometry of the structure depends on the magnitude and distribution of loading. In view of this, a permissible stress approach is adopted, together with a high factor of safety, between 5 and 10 for the fabric, and around 3 for the supporting structures (Bridgens *et al.*, 2004a: 24–25). There is no overall European code regulating the design of fabric structures at the time of writing. However, there are individual codes covering specific aspects of material design, testing, manufacture and performance. These codes are referred to in the document *European Design Guide for Tensile Surface Structures*, produced by TensiNet – a European Thematic network of practitioners and academics working in the field of tensile membranes.

Fabric structure and properties

Fabrics for tensioned membranes can be made from natural fibres such as cotton (canvas fabric), ceramics (glass fibre), and synthetic organic fibres, such as polyester. The base material is a woven fabric made of threads running longitudinally (warp direction) and at right angles to it (weft direction). There are two basic types of weave used: plain or basket weave, and Panama bond (Houtman, 2003: 183). The plain weave is shown in **Figure 1**; Panama bond is essentially the same weave, but achieved using more than one thread at a time. Thus, common descriptions, such as '2-2 Panama', or '3-3 Panama' indicate that the weaving operations were completed, respectively, with 2 and 3 threads at a time. The density of weave affects the strength of fabric and the relevant codes of practice (**Table 1**) specify this density in terms of 'number of yarns

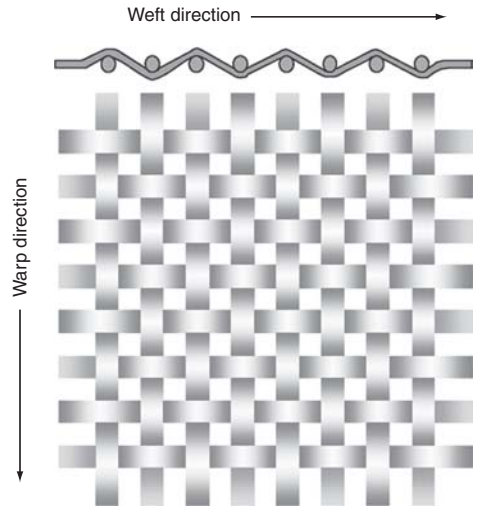

Figure 1 Plain or basket weave

per cm'. In this case, a '12 × 12 Panama' indicates that one centimetre of fabric contains 12 warp and 12 weft threads. Panama bond is stronger than basket weave, because of the multiple threads used but, ultimately, it is the type of fibre in combination with the type of weave that determines the fabric strength.

Strength and stiffness of fabrics

The strength of coated fabrics is determined mainly by the strength of their constitutive yarns. As can be seen from **Figure 1**, there are two basic directions of weave: warp and weft. Because fabrics are generally very thin (around 1 mm thickness), their stiffness and strength are not quoted in the usual stress units, but given in kN per metre width. This avoids a possible error magnification that would otherwise occur when dividing the load per metre by a small quantity (thickness). Sometimes units of daN/mm ((Newtons × 10)/mm), or N per 5 cm are used. The latter indicates that the result is taken directly from tensile tests on samples of fabric 5 cm wide. This form of quoting material properties is clearly not sufficient to identify individual fabrics and, therefore, fabrics are divided into several categories, or types (**Tables 1** and **2**), depending on their weight, strength, yarn density, density of weave, etc. The two key factors in these classifications are the weight and strength of fabric.

Both strength and stiffness of the fabric vary, depending on the direction weave and the loading regime. All architectural fabrics (except for foils, discussed under ETFE foils) exhibit anisotropic behaviour, i.e. they have different stress–strain characteristics in warp, weft and bias directions. The warp, however, is generally the stiffest direction (Hu, 2004: 113) and this is usually taken into account in the design of patterning of fabric membranes.

Material testing

The value of the breaking strength of fabrics is based on the *strip tensile test* performed on a clean, new and dry cloth. Uniaxial tests are more readily available and less expensive than biaxial ones, but they do not reflect the true state of stress in the fabric. In biaxial tests, fabrics exhibit strengths approximately half that value. Their strength is further affected by temperature, moisture penetration and creep.

Figure 2 shows typical, short-term, stress–strain curves from uniaxial and biaxial tests respectively.

Although most European standards for uniaxial testing are based on 5 cm width samples, the latest recommendations from Germany (Blum *et al.*, 2004: 295) advocate the use of 10 cm samples, and a test procedure given in

Type	1	2	3	4	5
Surface weight: g/m^2					
French design guide	720	1000	1200	1400	2000
WG Messe Frankfurt	800	900	1050	1300	1450
Yarn linear density: dtex					
French design guide					
WG Messe Frankfurt	1100	1100	1670	1670	2200
Tensile strength warp/weft: kN/m					
French design guide	60/60	84/80	110/104	120/130	160/170
WG Messe Frankfurt	60/60	88/79	115/102	149/128	196/166
Trapezoidal test warp/weft: N					
French design guide					
WG Messe Frankfurt	310/350	520/580	800/950	1100/1400	1600/1800
Yarn number per cm warp/weft					
French design guide					
WG Messe Frankfurt	9/9	12/12	10.5/10.5	14/14	14/14
Minimum prestress: kN/m	0.7	0.9	1.3	1.6	2.0

Table 1 Classification of PVC-coated polyester fabrics (adapted from Table 2, Blum *et al.*, 2004: 230, and Balz and Dencher, 2004: 192)

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Type	G_1	G_2	G_3	G_4	G_5	G_6	G_7
Tensile strength warp/weft: kN/m	26/22	43/28	70/70	90/72	124/100	140/120	170/158
Filament diameter: micrometre	9	6	3	6	3	3 or 6	3
Surface weight: g/m^2	500	420	800	1000	1200	1500	1600
Trapezoidal tear warp/weft: N			300/300	300/300	400/400	500/500	450/450
Prestress: kN/m	2.5% to 6% of the average tensile strip test						

Table 2 Classification of PTFE-coated glass fabrics (adapted from Table 3, Blum *et al*., 2004: 230, and Balz and Dencher, 2004: 193)

EN ISO 1421 and EN ISO 13934-1) in order to achieve more consistent results.

Figure 2(b) shows a typical biaxial response of one of the most common architectural fabrics, which is the PVC-coated polyester. The results are shown after several cycles of loading and for a ratio of warp : weft loading of 1 : 1. At a low level of straining (bottom part of the graphs) a phenomenon, known as 'crimp interchange' takes place, in which the generally straight warp fibres, under the influence of tension forces, undergo crimping

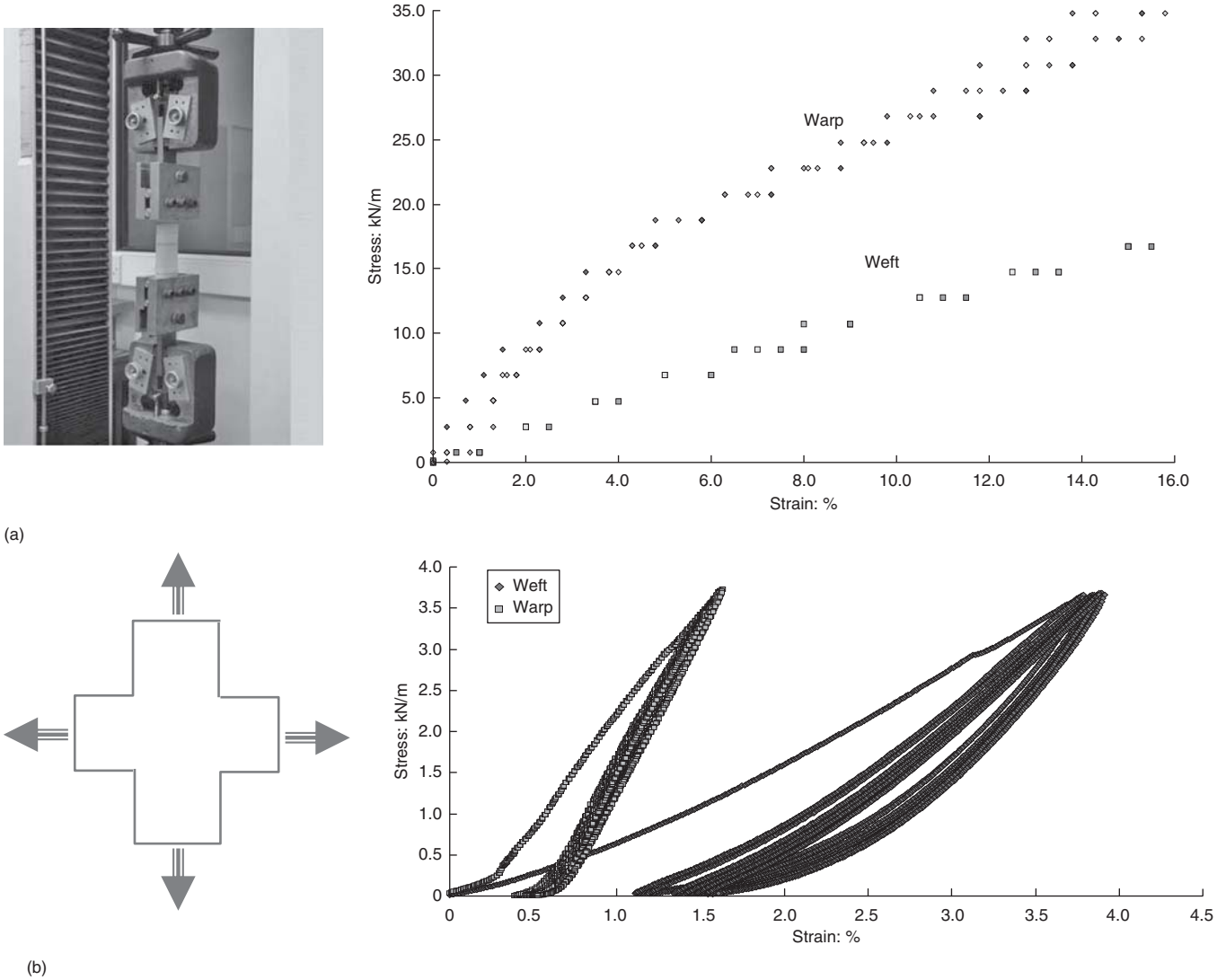

Figure 2 General stress–strain characteristics for PVC-coated polyester, obtained from (a) uniaxial *strip tensile* tests; (b) biaxial tests (1 : 1 ratio of warp : weft loading)

due to the interaction with the weft fibres. This is marked by a relatively flat part of the curve. The ratio of warp:weft loading affects the crimping and stiffness of the yarns and can lead to an apparent negative straining of yarns subjected to a lower level of loading. This effect is documented by Wakefield (2004: 106–107), using Teflon-coated glass-fibre fabric (the second most common architectural fabric on the market).

The generally non-linear and anisotropic properties of fabrics make it difficult to establish a single function describing their response to loading. The scatter of data requires statistical analysis to be able to predict the trend. For these reasons, Bridgens *et al.* (2004b: 32) advocate a departure from plane stress analysis towards a direct measurement of stress–strain response through biaxial tests.

One of the main difficulties in the design of fabric structures is ensuring that the prestress level in a fabric surface is kept at an appropriate level to prevent a possible loss of tension during the lifespan of the structure on the one hand, and allowing the material to stay within the elastic range of deformations, on the other. Under imposed loads, such as wind and snow, the stress in the surface can increase 6- to 8-, even 10-fold. For these reasons, and to ensure an adequate factor of safety against the propagation of cuts or flaws in the material, the design of tension membranes is aimed at keeping the prestress levels low: 1.3% to 6% of the strip tensile strength, depending on the type of fabric (Balz and Dencher, 2004: 192–193). The details of the recommended levels of prestress for the two most commonly used types of fabric are given in **Tables 1** and **2**.

The tear strength of fabric is obtained from a *trapezoidal test*. The procedure is illustrated in **Figure 3**. The test is performed on a trapezoidal piece of fabric of standard dimensions that has a short cut at a rightangle to one face, as shown. The fabric is clamped along two discontinuous lines and then torn apart during which process the load–extension relationship is recorded.

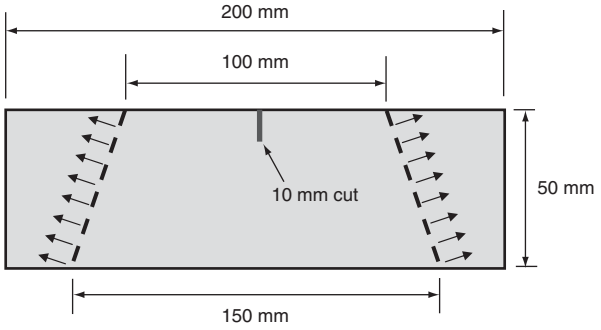

Figure 3 Trapezoidal tear test

Patterning

In full-size applications, it is not possible to construct a tensile membrane surface out of one continuous piece of material; the surface has to be divided into panels and assembled from unstrained, two-dimensional pieces of fabric. This process can be described mathematically as 'mapping' of a three-dimensional surface onto a flat plane, or a series of planes (**Figure 4**). It is known that such mapping, for undevelopable surfaces such as tensioned membranes, cannot be achieved without some distortion of the intended three-dimensional form. Therefore, efforts have to be made to minimise the errors involved. This can be achieved through accurate mathematical modelling and the appropriate choice of seam lines (Lewis, 2003: 158). Patterning requires good knowledge of fabric properties so that the stress levels in the material do not exceed the recommended values. The seam lines should follow minimum distances over the curved surface (known as geodesics) and the direction of cut should be aligned with the stiffer, warp direction of the weave.

Main categories of architectural fabrics

Base fabrics are treated with layers of polymer coatings to protect fibres against abrasion, improve watertightness of the surface, enhance self-cleansing properties, add flame-retardant quality, and protect the yarns against UV light. There is an enormous range of fabrics currently on the market, but the most commonly used are PVC-coated polyester and Teflon-coated glass fibre. The fuller range includes the following:

- PVC (polyvinyl chloride) – coated polyester
- PVC-laminated polyester
- PTFE (polytetrafluoroethylene) – coated glass fibre, also known as Teflon-coated glass. Teflon is a trade mark for PTFE materials supplied by DuPont
- PTFE – laminated glass fibre
- woven PTFE (Tenara) fabric – coated and uncoated
- silicone-coated glass fibre
- ETFE foils
- HDPE – high density polyethylene.

Samples of some of the fabrics are shown in **Figure 5**.

PVC-coated polyester

PVC-coated polyester has an estimated lifespan of 15 years and offers a range of high-strength fabrics, graded into five types (Blum *et al.*, 2004: 230). Type 1 is the lightest fabric (720–800 g/m^2) and type 5 is the heaviest (1450–2000 g/m^2). Further classifications within each type include:

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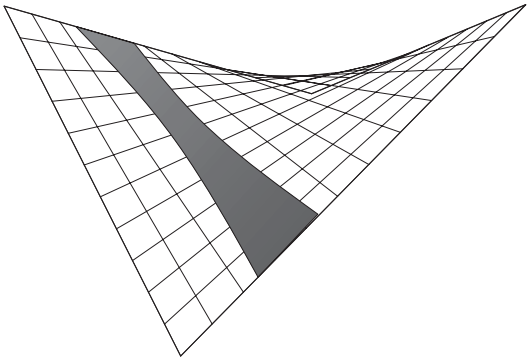
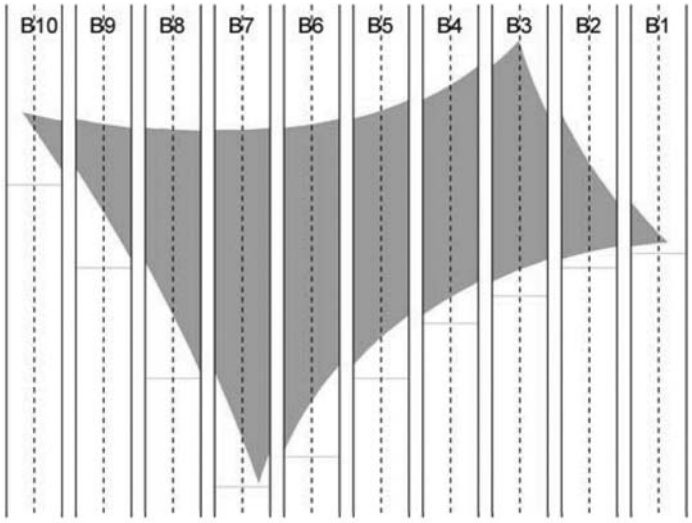


(a)



(b)

(c)

Figure 4 (a) Membrane surface; (b) geodesic cutting pattern; (c) membrane manufacture from cutting pattern panels (courtesy of Canobbio S.p.A)

- yarn linear density
- tensile strength (warp/weft) in kN/m
- trapezoidal tearing test (warp/weft)
- yarn number per cm (warp/weft)

and are given in **Table 1**. They are the recommendations of a Working Group at Messe Frankfurt and a French Design Guide.

Because individual yarns are very light, their weight, given as linear density, is quoted in 'dtex' – a weight in grams per 100 000 m.

The variety of types of PVC-coated polyester fabrics meets a wide range of structural requirements. The material is soft, flexible and comes in a variety of colours. It can easily fold up without damage to the surface, which makes it suitable for construction of convertible and transportable structures. The material is sealed with a radio-frequency welder, hot air or hot wedge welder and stitching.

The PVC coating is inherently flame retardant, but to fulfil the flammability criteria, it contains flame-retardant additives.

Among the fabric's undesirable qualities are: dirt retention, degradation under UV light causing cracking in the plastic layer and deterioration of the base material, and creep.

In order to overcome the dirt retention problem and improve durability of the material through a greater protection against UV light, a number of different topcoats have been developed. The basic topcoat is acrylic lacquer; a thin liquid layer applied to the surface of the PVC-coated polyester (http://www.canobbio.com; http://www.Ftlstudio.com). More expensive, but more durable topcoats are fluoropolymers, which include polyvinyl fluoride (PVF) and polyvinylidene fluoride (PVDF). However, these particular topcoats interfere with RF (radio-frequency) welding and, therefore, have to be ground off to enable the sealing of the joint between adjacent fabric panels (Armijos, 2008: 18). A weldable version of PVDF is available on the market, but the material has lower resistance to soiling. It is usually combined with acrylics to reduce cost (Houtman, 2003: 184). Where the exposure to UV light is particularly great, laminated Tedlar® (DuPont™ PVF film) is used to protect the coated fabric underneath. It is not a liquid top coat, but a film layer bonded to the PVC during the manufacturing process (Pudenz, 2004: 50).

Creep of PVC-coated fabrics translates to about 50% of prestress being lost after the first 10 days of installation and then, exponentially, after 100, 1000 days, etc. Hence, re-stressing procedures need to be implemented at roughly these time intervals.

It is now possible to recycle the material and recover both the polyester and PVC for further use (http://www.ferrari-textiles.com).

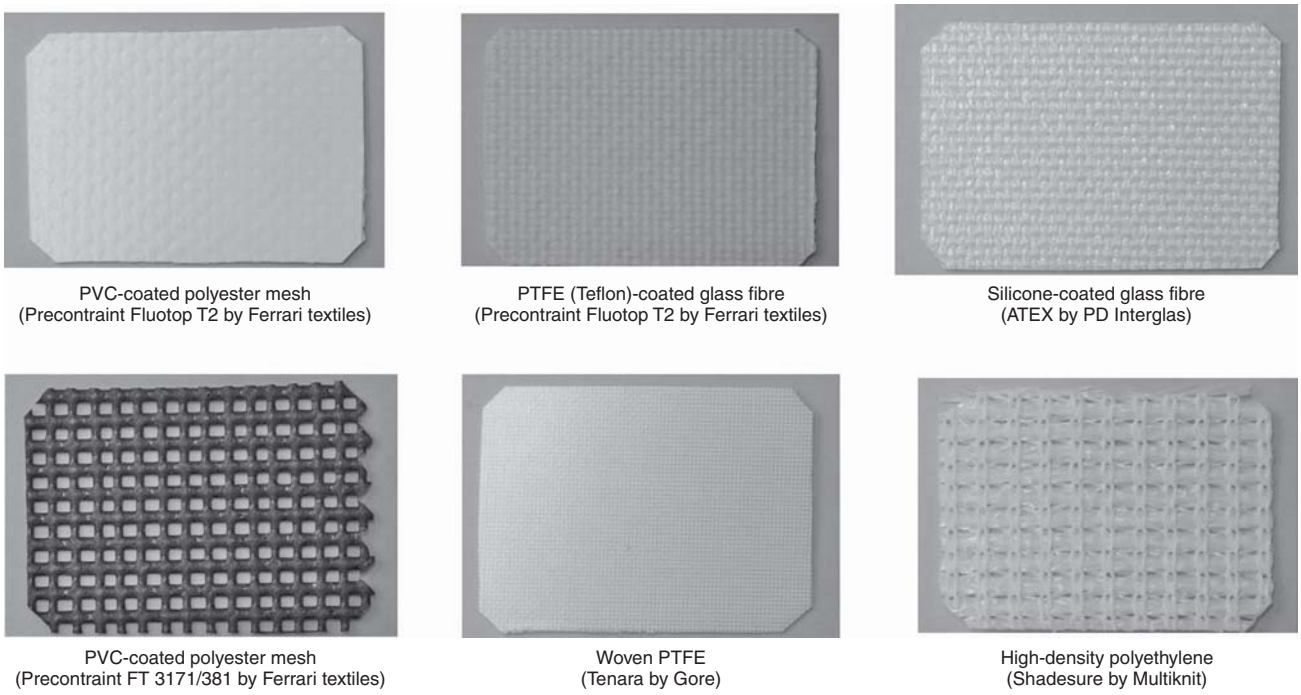


Figure 5 Images of samples of architectural fabrics

Research aimed at improving mechanical properties of the material led to the introduction of '*Precontraint*' fabric by Ferrari. The process involves prestressing both warp and weft polyester fibres prior to coating the material both sides (Lewis, 2003: 34–35). As a result, a more isotropic material is produced, but of greater stiffness than ordinary PVC-coated polyester.

Practical applications of PVC-coated polyester range from temporary to semi-permanent enclosures and shades. Examples of finished structures are given in **Figures 6** and **7**.

PVC-laminated polyester fabrics

PVC-laminated polyester fabrics consist of two or more layers of fabric joined together by water-based adhesive and subjected to heat and pressure to create a single ply. They have a shorter lifespan and are cheaper than the coated polyester and are used mainly as temporary struc-

Figure 6 Canopies in Parque Expo'98 in Lisbon, Portugal. Architect: M. Salgado; design: IPL Ingenieurplannung Leichtbau; membrane structure: Canobbio S.p.A. (courtesy of Canobbio S.p.A.)

Figure 7 Bandstand structure for the annual Kongsberg Jazz Festival, 2006. Architect: A.S. Snohetta; design: Airlight Ltd; membrane design: Form TL; membrane manufacture and construction: Canobbio S.p.A. (courtesy of Canobbio S.p.A.)

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tures, in the form of awnings and canopies (Armijos, 2008: 18).

Teflon-coated glass-fibre

Teflon-coated glass fibre is the most durable of fabrics on the market and the first choice for more permanent structures. The material is classified into seven types, according to its weight and tensile strength. The lightest type has weight of 500 g/m^2 and the heaviest 1600 g/m^2. The corresponding tensile strengths (expressed as a warp/weft ratio) are 26/22 kN/m and 170/158 kN/m, respectively. **Table 2** gives detailed information for each category of fabric.

PTFE-coated glass fibre has been in use since 1973, so it is known to have a lifespan of over 30 years (although manufacturers provide a guarantee for 15 years). Its natural, off-white colour changes to white within two months from installation. Because of its hydrophobic properties, PTFE gives excellent protection to the textile fabric, since glass filaments lose their tensile strength in contact with humidity. Further enhancement of impermeability, fungal resistance and weldability of the material is provided through FEP (fluoroethelynepropylene copolymer) topcoat. During welding, FEP film is used between layers of the fabric to secure a strong bond (Gipperich *et al.*, 2004: 251).

PTFE-coated glass fibre has translucency of up to 25% and its other attributes include self-cleaning properties, little creep and, therefore, minimum maintenance. The material shows superior resistance to tear compared with other architectural fabrics on the market. PTFE is totally resistant to UV and IR radiation and the membranes show no signs of ageing or brittleness due to these environmental factors (Blum *et al.*, 2004: 228).

The main disadvantage of the fabric is its brittleness; the material is prone to damage during transport and handling on site. Poor flexing properties and a tendency to self-abrasion (compared with PVC-coated polyester) impose greater demands on the installation process, detailing of connections, fabrication and design of the supporting structures. The installation process and tensioning of a medium-size membrane needs to take one to two weeks as opposed to a few days (in the case of PVC-coated polyester) to allow the material to relax gradually into the load regime. Care has to be taken at the design stage of the membrane, as the material is known to be less 'forgiving' to inaccurate cutting patterns.

Illustrative example

Figure 8 shows the Erasmus Station in Brussels where part of the Teflon-coated glass-fibre roof failed two weeks after installation and was replaced by more flexible PVC-coated polyester. It can be seen that the replaced panels represent a more aesthetic-looking cutting pattern.

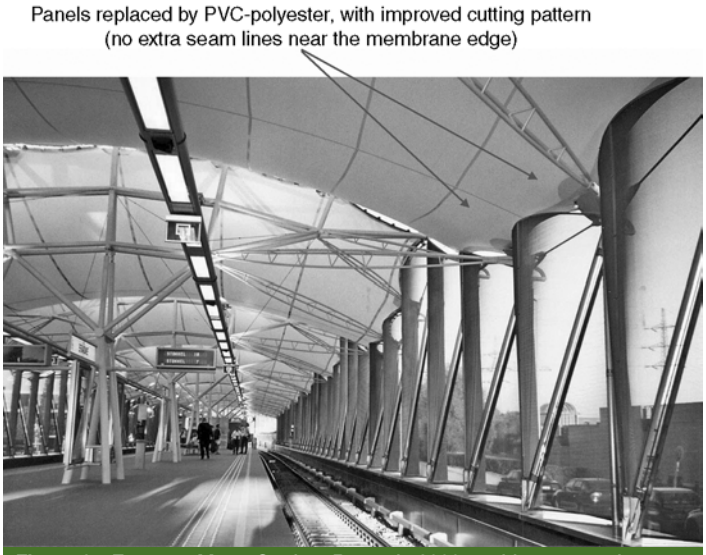

Figure 8 Erasmus Metro Station, Brussels 2003; architecture and engineering: Samyn and Partners

The roof consists of 11 membrane panels, each supported by two metal trusses. In the original design, the trusses were to support a more conventional roof and the decision to use fabric was taken late – after the design of the supporting structures was completed. It is reported (Samyn, 2003: 253) that part of the roof failed due to the speed of erection. However, further problems, such as the cloth corners having a tendency to move to a 'wrong' position and the use of high prestress (10 kN/m) would indicate a possibility of other contributing factors, such as an inaccurate prediction of shape of the membrane at the form-finding stage, inadequate cutting pattern, and inappropriate choice of supports. Had the architectural choice been to use a tensile membrane from the start, more flexible boundary supports, designed with the canopies in mind, might have prevented the failure.

Under fire, PTFE is known to give off supertonic emissions and, at one time, doubts were expressed about its suitability as a coating material. These doubts, however, were due to results based on laboratory tests simulating extremely high temperatures and no ventilation – an unlikely occurrence in real situations. Since then, the material has passed many international flammability tests and, currently, there is no restriction on its use in construction, provided such extreme conditions are unlikely to occur. It is important to note that the European directive 89/106/CEE defining the rules for fire safety of building construction products is not mandatory for materials used in tensile membranes, but is recommended to manufacturers as good practice. New methods for evaluation of reaction to fire (EN 13501-1: 2000) have recently been designed (Blum *et al.*, 2004: 320–321).

Practical applications of PTFE-coated polyester include permanent, mid-size and large-scale structures. **Figures 9**

Figure 9 The Millennium Dome (O_2 Arena), 2000; architect: Richard Rogers Partnership; structural design and detailing: Buro Happold; membrane patterning: Birdair

and **10** show examples of membrane structures realised in the material.

PTFE-laminated glass fibre

PTFE-laminated glass fibre contains a transparent fluoropolymer film laminated to each side of the fabric. This is a relatively new material, developed in the late 1980s (Pudenz, 2004: 51). It offers a greater degree of translucency, of up to 50%, while maintaining a similar strength to that of PTFE-coated material for medium-size membranes.

Figure 10 Mueller Offenburg, 1999. Project Executive: IPL; membrane installation: Canobbio S.P.A. (courtesy of Canobbio S.p.A.)

PTFE fabric

PTFE fabric, also known under the name 'Tenara', is an uncoated material woven from expanded PTFE fibre. Its main attributes are: high translucency (up to 40%); durability, flexibility, immunity to UV light, and resistant to acids, alkaline solutions and organic solvents. Originally developed as an open weave, the fabric was not waterproof. Its flexibility, however, made it particularly suitable for retractable structures (**Figure 11**).

Coated PTFE fabric

Coated PTFE fabric represents a fairly recent development. It uses PTFE coating on top of the base fabric, making it 100% PTFE. The material has lower tensile strength than PTFE-coated glass fibre or PVC-coated polyester, but superior tear resistance. Being new on the market, the material needs to withstand the test of time.

Silicone-coated glass fibre

Silicone-coated glass fibre is an inexpensive alternative to PTFE fabric, with a similar lifespan. The material is more flexible than most architectural fabrics and hence not subjected to brittle failures. It has very high tear strength, high translucency (25–42% depending on weight of fabric) and does not emit toxic fumes during burning. The reason why the industry has not readily embraced the fabric is two-fold: first, the fabric had a tendency to attract

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Figure 11 Mosque in Medinah, Saudi Arabia, 1992. Convertible umbrellas 17 × 18 m. Design: Sonderkonstruktionen und Leichtbau (SL), Stuttgart (courtesy of SL)

dirt and, second, it had poor seaming technology – in some early applications the silicon seams failed. However, the material is making a comeback due to the development of new, dirt-repellent topcoats and improved seaming technology. The new seaming process allows the fabric to be welded in a manner similar to that of PVC- and Teflon-coated materials and produces more consistent quality of seams.

Practical applications of silicone-coated glass fibre range from temporary to permanent structures, which can be medium or large scale. The material is particularly suited to situations where a high level of translucency is required.

Figure 12 shows one of the largest music theatres – a concrete structure wrapped in 12 000 square metres of silicone-coated glass fibre – selected because of its translucency.

As stated earlier, polyester and glass fibre fabrics are the two most popular fabrics on the market. A qualitative comparison of their properties is given in **Table 3**. Additional comparisons are given in **Table 4**.

Figure 12 Music Theatre Zenith, 2007. Architect: Massimiliano Fuksas; membrane and project details: Form TL, Membrane construction: Canobbio S.p.A. (courtesy of Canobbio S.P.A.)

ETFE foils

ETFE foils are made out of a copolymer of ethylene and tetrafluoroethylene. The material main attributes are: low weight (~1% of the weight of glass), self-cleaning properties, translucency of up to 90%, and good resistance to fire, chemicals and environmental factors (Pudenz, 2004: 51: Houtman, 2003: 190; Armijos, 2008: 19).

Although the raw ingredients used in the production of ETFE include natural resources, such as gas, oil and other minerals, the quantities used are relatively small per square metre of area, and the material is recyclable. The manufacturing process does not involve significant use of additives. The inflation units within the structure consume energy, but the high translucency of the material compensates for this in reduced artificial lighting. Therefore, the material and the type of structure are viewed as environmentally friendly (Jones, 2000: 93).

Foil comes in thicknesses varying from 50 µm to 250 µm. The thickest foil has a tear strength approximately equal to 10 kN/m (Houtman, 2003: 190), which is 1/6 of the strength of type 1 PVC-polyester. Unlike other architectural fabrics,

	Polyester fabric			Glass fibre fabric	
Coating	PVC	PVC	PVC	PTFE	Silicone
Top coating	Acrylic	PVF-lamination	PVDF-merging		
Expected lifetime	8–10 years	12–15 years	12–15 years	>30 years	>30 years
Ageing	Average	Good	Good	Very good	Very good
Self-cleaning	Average	Good	Good	Very good	Average
Transparency	Good	Good	Good	Good	Very good
Fire-retardant	Good	Average	Good	Very good	Very good
Foldability	Very good	Average	Average	Bad	Average

Table 3 A qualitative comparison of polyester and glass fibre fabrics (adapted from Table 2, Houtman, 2003: 190)

	PVC-coated polyester fabrics	PTFE-coated glass fabrics	Silicone-coated glass fabrics	PTFE-coated PTFE fabrics
Tensile strength warp/weft: kN/m	115/102	124/100	107/105	84/80
Fabric weight: g/m^2	1200 (type 3)	1200 (type G_5)	1100	830
Trapezoidal tear warp/weft: N	800/950	400/400	960/700	925/925
Visible light transmission: %	10–15	10–20	<80	19–38
Flexibility/crease recovery	High	Low	High	High
Cleaning	Easier with top coats	Self cleaning	Self cleaning	Self cleaning
Seam technology	High frequency welding	Thermal welding	Vulcanisation	Stitching
Life span: years	>15–20	>25	>25	
Cost	Low	High	High	High

Table 4 Additional comparative properties of architectural fabrics (adapted from Table 4, Blum *et al*., 2004: 231)

Thickness: microns	Weight: g/m^2	Tensile strength L/T: N/mm^2 (DIN 53455)	Tensile strain L/T: %	Tear strength L/T: N
50	87.5	64/56	450/500	450/450
80	140	58/54	500/600	450/550
100	175	58/57	550/600	430/440
150	262.5	58/57	600/650	450/430
200	350	52/52	600/600	430/430
250	437.5	140/140	>300/300	>300/300

Note: L/T – Longitudinal/Transverse direction

the material is isotropic but exhibits a bilinear stress–strain relationship. Its elastic properties are summarised in **Table 5**.

This material is used in pneumatic structures, alternatively known as 'inflated pillows' or 'cushions'. The foil is supported by a constant air flow supplied by air pumps and an emergency backup system. Humidity and temperature controls plus filters are installed to prevent moisture and dirt from getting inside the cushions. Practical applications of ETFE are shown in **Figures 13** and **14**.

HDPE – high-density polyethylene

HDPE has an estimated lifespan of 10 to 12 years. The fabric is 100% recyclable and can be manufactured for shading only, or engineered for complete water protection (Armijos, 2008: 19). Its other attributes are high tensile strength and UV absorption. Coated polythene produces higher strength-to-weight ratio than most architectural fabrics. **Figure 15** shows a practical application of the fabric.

Factors to consider in the choice of fabric

The choice of architectural fabric is dependent on numerous considerations, which include: material properties, the type of tensile structure, functional requirements, intended durability, and manufacturing considerations. These will now be reviewed in turn.

Strength and safety

Weight of the material

The weight of architectural fabrics is very low, varying from 0.2 kg/m^2 to 1.5 kg/m^2 depending on their type and class. In the case of the very light materials, such as foils, it is important to consider the effects of wind and drifting snow on the behaviour of a fabric structure.

Figure 13 Eden Project, Cornwall, 2001; architect: Grimshaw; climatic analysis: Arup Associates; structural design: Anthony Hunt Associates; construction: McAlpine Joint Venture (courtesy of Ben Foster)

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Figure 14 Alnwick Garden Pavillion, 2006; architect: Hopkins Architects; engineer: Buro Happold; membrane structure: Hightexworld (courtesy of Paul Tyagi)

Tensile and tear strengths

As can be seen from **Tables 1** and **2**, strength of fabrics is closely related to their weight. Manufacturers' specifications of mechanical properties of fabrics cannot be relied upon in isolation – they need to be confirmed through testing, but even then there is a significant margin of uncertainty associated with the actual fabric strength. The long-term properties are a subject of continual research.

Figure 15 Covered parking for Classic Hummer, Dallas, 2006; design and fabrication: Vehicle Protection Structures (courtesy of USA Shade and Fabric Structures)

Flammability

While the new methods for evaluation of reaction to fire (EN 13501-1) await implementation for fabric structures, a low flammability, according to DIN 4102 (1998) is generally the minimum requirement.

Function and aesthetics

Acoustic and thermal properties

Architectural fabrics, particularly foils, are characterised by a low sound absorption. However, recent technological advances have led to the development of micro-perforated transparent foils that are characterised by excellent acoustic absorbency (Pudenz, 2004: 60).

Acoustic insulation can be generally improved by using a multi-layer fabric construction. The same solution can be used to provide an increased thermal insulation. Although the materials have low thermal conductivity, their thickness, of only 0.2–1.5 mm, cannot provide an adequate thermal insulation. Where this is a problem, either mineral, or transparent insulation may be added to the cavities between fabric layers.

Light transmission, reflection and absorption

Architectural fabrics do not transmit light evenly across the colour spectrum. The translucency of fabrics has been exploited to reduce the need for artificial lighting. The light that gets transmitted is diffused (scattered in all directions) and natural. It has the advantage of being soft and flattering, but the lack of shadow may hinder the recognition of shapes and textures within the enclosure (Huntington, 2003: 156–158).

Temperature range

The temperature range of the environment does place some restrictions on the choice of fabrics. PVC-coated polyester should not really be used in areas of sub-zero temperatures all year around. Once installed, PTFE-coated glass fibre is insensitive to fluctuations in temperature. However, its installation at temperatures below 5°C should be avoided (Pudenz, 2004: 63).

Colour range

PVC-coated polyester offers by far the greatest range of colours. In the case of PTFE-coated glass fibre, there are only a few colours available, due to the high temperatures used in the production of the material and, consequently, the need for colour pigment to be extremely resistant to heat. In selecting the colour, the effects on the lighting conditions underneath the membrane have to be taken into account.

Maintenance

Self-cleaning properties of the fabric are essential in maintaining the required translucency and aesthetic appearance. In this respect, the PTFE-coated glass fibre is superior to silicone- and PVC-coated fabrics. Although fluoropolymer topcoats improve the dirt retention of PVC significantly, at the same time they interfere with RF welding and increase fabric stiffness.

The design details of tensile membranes need to take into account maintenance issues by allowing easy access to the fabric for periodic cleaning.

Ease of construction and suitability for a particular application

Flexibility

As stated earlier, PVC-coated polyester is more pliable than PTFE-coated glass fibre. This property makes it suitable for application in retractable structures and structures of generally complex form that may require any inaccuracies in cutting patterns to be accommodated by the fabric stretch.

Assembly process

PTFE-coated glass fibre requires a relatively slow process of installation and tensioning on site. ETFE foils require particularly careful handling at the manufacture and assembly stages, in which a special machine welding process needs to be employed together with the use of non-conventional structural sections for connecting the foil panels.

Durability and sustainability

Durability of architectural fabrics depends on their resistance to environmental factors, namely UV radiation and response to chemicals in the atmosphere. In convertible and transportable structures, the limiting factor is the resistance to abrasion and tolerance of bending/folding. The design details also play an important role here (Seidel, 2009).

Future trends

Architectural fabrics are a field of intensive research. A Working Group, called *Analysis and Materials*, has been set up within the TensiNet. Its purpose is to advance knowledge of the subject with regard to: the development of improved modelling and testing tools for fabrics, new materials, assessment of long-term performance, accurate determination of thermal, acoustic, and optical properties of fabrics, improved monitoring of fabric structures, and accurate wind loading prediction. The most extensive work within these themes is research into translucent insulation to improve sound absorption and thermal insulation of architectural fabrics.

Light-activated colouring of fabrics is one of the more recent developments. Using a printed coating of pigment particles that are activated by light, a white or natural colour membrane can be illuminated with a spectrum of colour and in a variety of shades (Pudenz, 2004: 61). Extremely thin, highly flexible and very light photovoltaic cells have now being developed, which are encapsulated in fluoropolymer coating and used on both ETFE foil and PTFE fabric. These new 'PV Flexibles' products come in rolls of 0.5 m width and 3 m length (maximum) and are particularly suited to multi-layer air cushions and large membrane structures.

Research into fabrics is benefitting from the cross-over technology in neighbouring textile industries (e.g. Tenara fabric by Gore, or smart fabrics by Eleksen (http://www.eleksen.com/). Three-dimensional weaves, used in the fabric for convertible car hoods, offer exciting possibilities for tensile membranes.

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Appendix A

Fire performance of commonly used building materials

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This appendix provides a quick source of reference for the design and performance in fire of a number of commonly used construction materials. Material behaviour at elevated temperatures is a complex phenomenon and the subject of a number of detailed textbooks, guidance documents, and national and international codes and standards.

Introduction

The information in the table that follows and the temperature dependent material properties that accompany it are provided as guidance to the anticipated performance of the materials in a fire situation and the associated measures that may need to be taken in order to achieve the required regulatory compliance with respect to a fire situation.

Although the *Manual* covers a wide range of construction materials, this appendix focuses on those materials most commonly used for structural frames. Performance in fire can be assessed in terms of reaction to fire and fire resistance. Reaction to fire deals with issues such as combustibility and the rate of heat release following ignition. Fire resistance is primarily concerned with the maintenance of the load-bearing and/or separating function for a duration sufficient to ensure safety in the event of a fire of building occupants, those in the vicinity of the building and the Fire and Rescue Service. The requirements of the building regulations in terms of both reaction to fire properties of materials and the fire resistance of load-bearing and non-load-bearing elements of construction are dependent on both the type of occupancy (related to the risk of ignition) and the height of the structure (related to the consequences of failure). Those materials most commonly used to provide the load-bearing elements for medium-rise and multi-storey construction are those that will therefore need to demonstrate the highest levels of performance.

In general, structural performance in the fire situation is related to survival in a standard fire test (BS 476-20: 1987; BS EN 1363-1: 1999). The guidance in the Approved Document (Communities and Local Government, 2007) is related to specific periods of fire resistance and the traditional means of ensuring compliance is to rely on prescriptive guidance based on results from standard fire tests. As it is clearly impractical to test every individual form of construction, most designers rely on the use of tabulated data, much of which is published in design guides or codes and standards (BS 8110-2: 1985; Morris *et al*., 1988; ASFP, 2002). The advent of the structural Eurocodes has made available a number of performance-based solutions to structural fire engineering design, based on calculation techniques that take into account the specific characteristics of the building under consideration. Those construction materials discussed here all have their own fire design part in the Eurocode system (BS EN 1993-1-2: 2005; BS EN 1992-1-2: 2004; BS EN 1995-1-2: 2004; BS EN 1996-1-2: 2005; BS EN 1994-1-2: 2004).

Some guidance is provided in terms of further reference for each of the main construction materials addressed, so that the reader may access more detailed information depending on the specific requirements of the project under consideration. One specific source of guidance which covers the performance in fire of all of the main construction materials is worth highlighting for its extensive database of elevated temperature material properties, guidance on the use of the relevant codes and standards, published research and structural fire engineering design case studies. The one-stop shop for structural fire engineering, set up and maintained by Professor Colin Bailey of the University of Manchester, is a free web-based resource for the construction industry and an invaluable source of information for the specialist and non-specialist alike.

Material	Sub group	General performance	Combustibility (BS Classifications and European Classifications)	Relevant standards (National and European)	Heat transfer characteristics	Strength/ Stiffness reduction characteristics	Protection requirements	Achievable fire resistance	Further reading/ References
Steel	Hot-rolled	Characterised by loss of stiffness and strength at elevated temperatures with the reduction particularly pronounced between 500 and 600°C	Non-combustible	BS 5950 Part 8 BSI, 2003/ BS EN 1993-1-2 BSI, 2005	Dictated by section factor (ratio of heated perimeter to cross-sectional area)	Refer to Figure 1 and 2	Fire resistance periods of up to 30 min for unprotected steel. Fire resistance periods of up to 60 minutes using partial protection	>240 min for fire resistance periods in excess of 60 min applied. Passive fire protection (PFP) normally required in the form of sprays, boards or intumescent coatings	BRE Digest 487 Part 2, Bailey, 2004
	Cold-formed	Loss of strength with temperature greater than for hot-rolled member by 10–20% [5]		BS 5950 Part 5 BSI, 1998/BS EN 1993-1-2 BSI, 2005			Often used for light gauged members where heat transfer to the structural member would be very rapid if unprotected. Fire resistance periods of up to 120 minutes with applied PFP	Up to 120 min fire protection generally in the form of boards covering light gauge steel joists and studs	SCI Publication 129, Lawson, 1993
Concrete	Siliciceous	Good performance in fire. May be prone to spalling when subject to combination of high moisture content, restraint to thermal expansion and rapid increase in temperature	Non-combustible	BS 8110 Part 2 BSI, 1985/BS EN 1992-1-2 BSI, 2004	Slow increase in temperature due to relatively low thermal conductivity (when compared to steel). Results in large thermal gradients through the depth of the member	Refer to Figure 3	Does not normally require additional PFP for use in buildings. Fire protection may be required for specific applications such as tunnel linings or use in petro-chemical industry	In excess of 240 min	BRE Digest 487 Part 1, Lennon, 2004
	Calcareous	Lower thermal conductivity than concretes made from siliceous aggregates so tend to be more stable at high temperatures		BS 8110 Part 2 BSI, 1985/BS EN 1992-1-2 BSI, 2004					
	Lightweight concrete (LWC)	Generally less conductive than more dense concrete's due to the presence of small air voids in the cured mix. Less susceptible to spalling due to a comparably higher porosity		BS 8110 Part 2 BSI, 1985/BS EN 1992-1-2 BSI, 2004					Fire safety of concrete structures, Lennon, 2004
	High-strength concrete (HSC)	Prone to explosive spalling resulting from a combination of thermal and pore pressures. Has low porosity/permeability which inhibits stress reduction from crack formation		BS EN 1992-1-2 BSI, 2004			Additional measures required which may involve use of sacrificial steel, traditional PFP or the use of polymeric fibres		High Strength/ High Performance Concrete, Konig *et al*., 2002

Timber	Solid timber	Large section timber may have substantial fire resistance due to formation of an insulating char layer	Combustible	BS 5268-4.1 BSI, 1978/BS EN 1995-1-2 BSI, 2004	Due to the combustible nature of timber, char layers develop on the outside of exposed members. The char layer is typically less conductive than the solid timber which protects the residual section for large durations. For small sections the char layer can engulf the entire cross section. Refer to Table 1	Refer to Figure 4	Heavy weight solid timber construction does not generally require protection due to the resistance provided by the char layer. Fire retardant chemicals are available for the treatment of timber, which reduce the ignitability of the timber. However it is unlikely that any treatment will stop the combustion of timber when exposed to fully developed fires	Up to 120 min	BRE Digest 487 Part 4, Bregulla, 2004
	Light timber frame	Little inherent fire resistance – generally used in conjunction with applied PFP in the form of boards		BS 5268-4.1 BSI, 1978/BS 5268-4.2 BSI, 1990/ BS EN 1995-1-2 BSI, 2004	Light timber-frame and timber-based composites typically utilise much smaller members/section sizes than solid timber constructions. The thermal properties of timber are identical whether used in light or heavy applications. However the member size and relationship between cross sectional area and heated perimeter result in more rapid increases in temperature in smaller sections. Additionally thick char layers cannot develop to protect the section due to its comparably small cross section		Additional PFP required for most practical applications usually in the form of gypsum or vermiculite based plasterboard, calcium silicate boards or other fire resisting materials. The fire properties of timber can also be modified using surface treatments such as intumescent coatings and impregnated inorganic salts	Fire resistance achievable is almost entirely dependant on the dry lining. Typically light timber-frame and timber composites can achieve 60 min of fire resistance when adopted with two layers of fire rated plasterboard (12.5 mm thickness per layer)	BRE Digest 487 Part 4, Bregulla, 2004
	Timber composites	As above		BS 5268-4.2 BSI, 1990/BS EN 1995-1-2 BSI, 2004					TRADA – Structural timber composites – Design guide 1, Mettem *et al*., 1996
Masonry		Excellent inherent fire resistance. May be prone to instability due to thermal bowing for slender walls	Non-combustible	BS 5628-3 BSI, 2001/BS EN 1996-1-2 BSI, 2005	Generally the thermal conductivity of most masonry units is low which results in low temperatures in the units for large durations. The presence of water results in a moisture plateau where the temperatures stay below 100°C until excess moisture evaporates	Refer to Figure 5	Does not generally require additional PFP	In excess of 240 min (refer to Table 2)	BRE Digest 487 Part 3 [18], BRE IP 21/88, Cooke and Morgan, 1998, BR 128, Morris *et al*., 1988

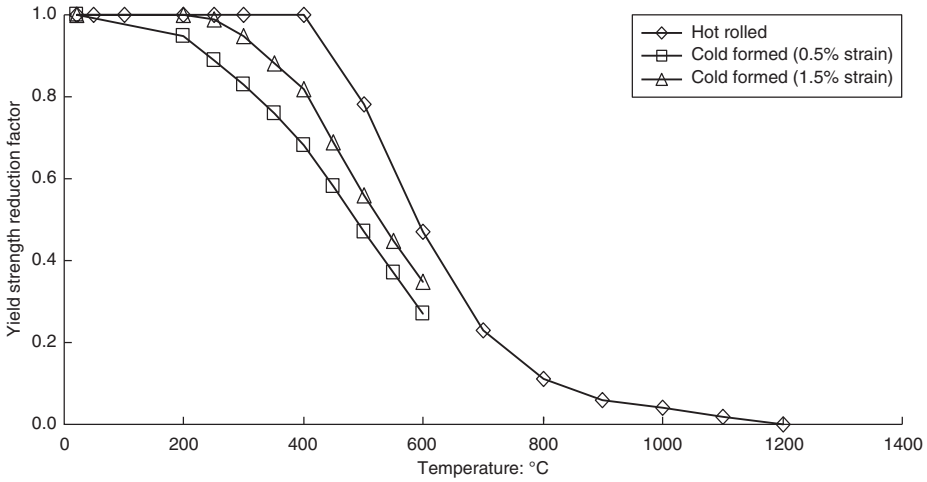

Figure 1 Reduction in yield strength with temperature (Steel) BSI, 2005; Lawson, 1993

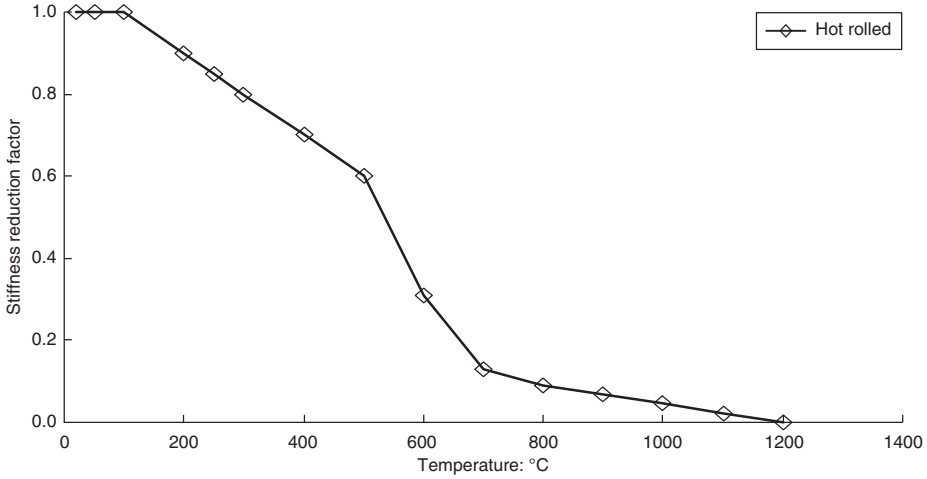

Figure 2 Reduction in elastic modulus with temperature (Steel) BSI, 2005

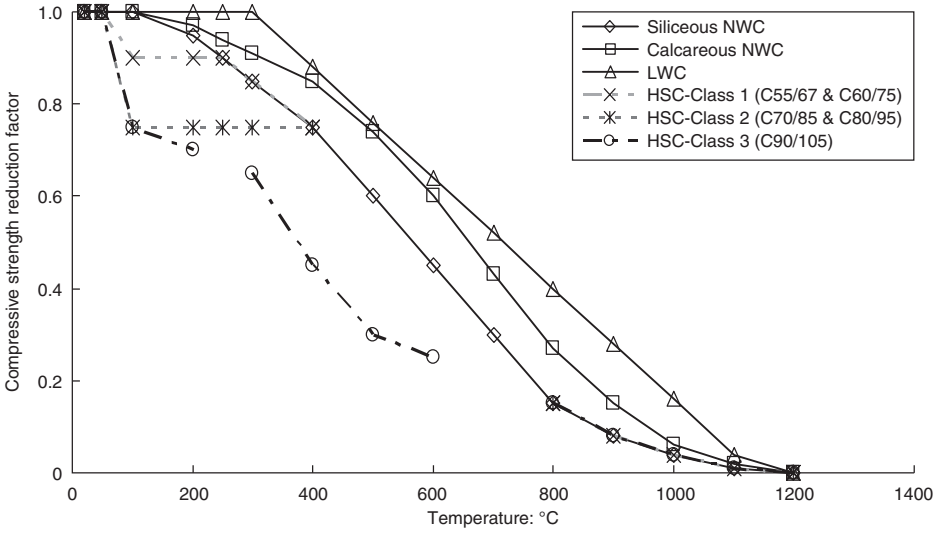

Figure 3 Reduction in compressive strength with temperature (Concrete) BSI, 2004

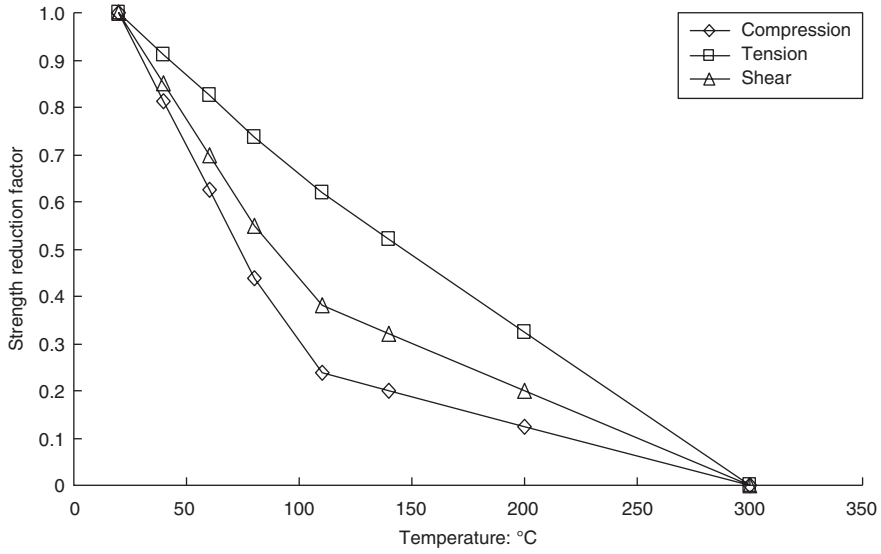

Figure 4 Reduction in strength with temperature (Timber) BSI, 2004

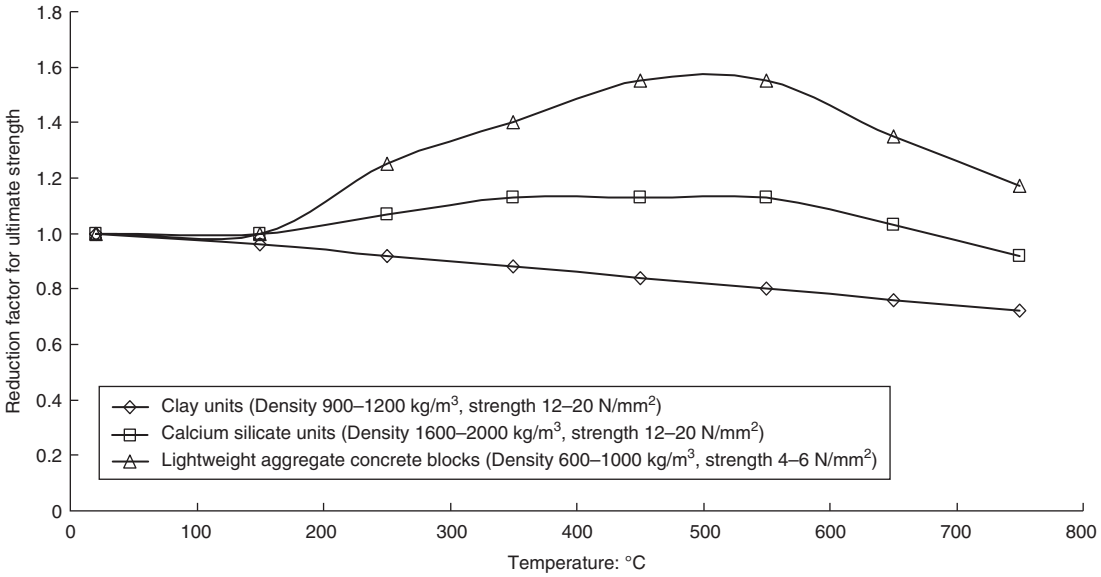

Figure 5 Change in compressive strength with temperature (Masonry) BSI, 2005

	1D charring rate under standard fire exposure: mm/min
(a) Softwood and beech	
Glued laminated timber (density $\geqslant$290 kg/m^3)	0.65
Solid timber (density $\geqslant$290 kg/m^3)	0.65
(b) Hardwood	
Solid or glued laminated hardwood (density of 290 kg/m^3)	0.65
Solid or glued laminated hardwood (density $\geqslant$450 kg/m^3)	0.5
(c) LVL	
Density $\geqslant$480 kg/m^3	0.65
(d) Panels (based on 20 mm thickness, density of 450 kg/m^3)	
Wood panelling	0.9
Plywood	1
Other wood based panels	0.9

Table 1 Charring rates under standard fire exposure (Timber) BSI, 2004

Nature of construction and materials	Masonry: Minimum thickness (mm), excluding any finish, for a fire resistance (mins) of:					
	30	60	90	120	180	240
Bricks of clay, brickearth or shale						
(a) without finish	90	90	100	100	170	170
(b) with 13 mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
Bricks of concrete or calcium silicate						
(a) without finish	90	90	100	100	190	190
(b) with 13 mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
Blocks of dense concrete						
(a) without finish	90	90	100	100	–	–
(b) with 13 mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
Blocks of lightweight concrete						
(a) without finish	90	90	100	100	140	150
(b) with 13 mm lightweight aggregate gypsum plaster	90	90	90	90	100	–
Blocks of autoclaved aerated concrete of density (480 to 1200 kg/m^3)						
(a) without finish	90	90	100	100	140	180
(b) with 13 mm lightweight aggregate gypsum plaster	90	90	90	100	100	–
Blocks of autoclaved aerated concrete of density (400 to 479 kg/m^3)	100	100	120	130	160	200

Table 2 Fire resistance of masonry units, Morris *et al*., 1998

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Appendix B

Guide to the recycled content of mainstream construction products: Waste & Resources Action Programme (WRAP)

doi: 10.1680/mocm.35973.0895

Introduction

Table B.1 identifies the range of recycled content for generic construction products and materials, giving estimates for recycled content. The tabulated values do not mean that there are individual branded products with recycled content exactly matching the standard, good and best practice.

In addition, the table does not indicate that all demand could be supplied with good or best recycled content – there may be constraints on manufacturing capacity and the supply of recovered materials (at least in the short term).

The data, therefore, provide guidance rather than a definitive or exact analysis. Buyers should ask suppliers for details relating to individual products.

The Building Research Establishment (BRE) and the Construction Products Association originally provided the data to WRAP. Some entries have been updated to take account of additional evidence of products available on the market – collected by AMA Research, Arup, Cyril Sweett, Davis Langdon, Faithful & Gould, and Scott Wilson. The table identifies three levels of recycled content: standard practice, good practice and best available.

- Standard practice (baseline or default, describing the lowest recycled content normally achieved).
- Good practice (higher level of recycled content readily available in the market and achieved at no additional cost).
- Best practice (the highest recycled content currently available, but potentially incurring additional cost).

This table has been reproduced with kind permission of WRAP (http://www.wrap.org.uk/construction).

Materials	Product	Recycled content: as a % of mass		
		Standard	Good	Best
Metals	Reinforcing steel		100	
	Steel section		60	
	Steel coil		15	
	Coated steel coil		15	
	Hot-dip galvanised steel coil		15	
	Stainless steel		75	
	Copper sheet		60	
	Copper cable and pipe		15	
	Copper alloys (brass, copper nickel)		95	
	Lead		80	
	Zinc		31	
	Aluminium extrusion		44	
	Aluminium sheet		73	
	Aluminium foil		20	
Board materials	Chipboard	65	70	90
	Fibre boards			
	Hardboard		60	
	Medium-density fibreboard (MDF)		60	90
	Other boards			
	Plasterboard	36	84	98
	Fibre-reinforced gypsum board	5	60	100
Glass	Float glass		10	
Brick and block	**Walls**			
	Dense block	0	50	93
	Lightweight block	50	80	93
	Aerated block	0	50	65
	Foamed glass block		66	
	Clay brick	0	10	30
Floor covering materials	**Carpet and backing**			
	Carpet with latex backing and latex foam (rubber)	0	25	
	Needle punch felt	0	20	25
	Carpet tiles			
	Generic carpet tile	0	50	100
	Underlay			
	Rubber underlay	0	90	100
	Natural fibre underlay (felt)		100	
	Other floor coverings			
	Linoleum	5	24	37
	Vinyl (PVC)	0	25	100
	Rubber	0	20	40
	Heavy-duty rubber floor	0	40	85
	Resin bonded (reconstituted) tiles	0	40	80
	Ceramic tiles and flags	0	9	20

Table B.1 Continued

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Materials	Product	Recycled content: as a % of mass		
		Standard	Good	Best
Ceiling materials	Mineral ceiling tiles	10	50	78
	Gypsum ceiling tiles	36	84	98
	Aluminium ceiling tiles		73	
	Steel ceiling tiles	15		25
Roof materials	**Cement-based tile**			
	Concrete tile	0	5	22
	Fibre cement	0	5	10
	Reconstituted (resin-bonded) slates	43	80	95
	Polymer-modified cement tiles		30	
	Metal sheet			
	Galvanised steel sheet		15	
	Organic-coated steel sheet		15	
	Stainless steel sheet		75	
	Copper sheet		60	
	Lead sheet		80	
	Zinc sheet		31	
	Aluminium sheet		73	
	Other			
	Rubber roof slate		99	
	Roofing felt	0	75	90
Cavity closure	PVC-U cavity closure systems	4	80	100
Aggregate	General fill in building and civils applications	0	25	100
	Highways reinstatement (utilities work)	50	80	100
	Pipe bedding (building work)	0	50	100
	SHW types 1 and 2 sub-base	0	100	
	Coarse aggregate in concrete	0	20	100
	Coarse aggregate in low-strength mass concrete	0	30	100
	Aggregates in hydraulic bound material (HBM) and cement bound material (CBM) sub-base and base course	0	60	100
	Aggregate in bituminous bound road pavements (off-site process)	0	10	40
	Aggregate in bituminous bound road pavements (in-situ and ex-situ cold process)		100	
Mortar	Standard mortar mix (includes binder and aggregate contents)	3	4	70
Concrete	Cast in-situ reinforced structural concrete – C25 or lower	15	32	90
	Cast in-situ reinforced structural concrete – C30 or higher	24	30	44
	Cast in-situ concrete, C25 or lower – no reinforcing	0	24	90
	Cast in-situ concrete, C30 or higher – no reinforcing	0	7	26
	Precast reinforced structural concrete	20	22	23
	Trench fill foamed concrete	0	40	95
	HBM and CBM	0	50	98
Thermal and acoustic insulation	Mineral wool (also termed rock wool)	25	50	
	Glass wool	30	50	80
	Foamed glass		66	
	Cellulose fibres	80	85	100
	Polyethylene	1	4	
	Polystyrene expanded (EPS)	0	25	
	Rubber and elastomeric	18	60	98

Table B.1 Continued

Materials	Product	Recycled content: as a % of mass		
		Standard	Good	Best
Other infrastructure	Rubber highway drainage material		85	
	Plastic lumber	83	96	100
	Plastic sheet piles		100	
	Geotextile mats	0	90	
	Asphalt containing glass (highway application)	10	15	30
	Rubber safety surfacing		100	
Rain water goods	Aluminium		73	
	Stainless steel		90	
Pipes	Copper plumbing pipes		15	
	PP soil and waste pipes	0	75	100
	HDPE land pipe	20	45	100
	PVC soil and waste pipes	0	10	
	PVC road drainage and land drainage pipe	0	50	100
Plaster	Lime-based	0		70
	Gypsum-based	0	45	95
Landscaping materials	Growing media, compost, mulch, soil improvers	0	10	100
	Paving			
	Concrete kerb stones	3	20	75
	Concrete paving slabs	3	20	75
	Concrete paving blocks	5	50	90
	Concrete reconstituted stone paving blocks	5	40	60
	Glass paving blocks		100	
	Brick paving	0	10	30
Electrical systems	**Cable**			
	Copper		15	
	Cable management – trunking and ducting			
	HDPE	0	60	75
	PP	0	60	75
	PVC	0	30	75
	Aluminium		73	
	Galvanised steel		7	

Note:

1 This table gives estimates for recycled content. The tabulated values do not mean that there are individual branded products with recycled content exactly matching standard, good and best practice. In addition, the table does not indicate that *all* demand could be supplied with good or best recycled content – there may be constraints on manufacturing capacity and the supply of recovered materials (at least in the short term).

2 The data provide guidance rather than a definitive or exact analysis. Buyers should ask suppliers for details relating to individual products.

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